FEASIBILITY REPORT

SECTION 103
SHORE PROTECTION AND STORM DAMAGE
REDUCTION STUDY

WEST SILVER SANDS BEACH
EAST HAVEN, CONNECTICUT

MAY 1996

US Army Corps
of Engineers
New England Division

SHORE PROTECTION AND STORM DAMAGE REDUCTION STUDY

WEST SILVER SANDS BEACH, EAST HAVEN, CT

FEASIBILITY REPORT

MAY 1996

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW ENGLAND DIVISION
WALTHAM, MA

WEST SILVER SANDS BEACH, EAST HAVEN, CT
SHORE PROTECTION AND STORM DAMAGE REDUCTION STUDY
FEASIBILITY REPORT

EXECUTIVE SUMMARY

This report was prepared under the authority of Section 103 of the River and Harbor Act of 1962, as amended (PL 87-874). Section 103 authorizes the study, design and construction of civil works for the prevention of shoreline storm damages. The Section 103 study of the West Silver Sands Beach area was requested by the town of East Haven, CT as a result of storm damages sustained in the area during severe coastal storms. The purpose of the study is to investigate the existing and potential coastal damages occurring in the area and formulate and analyze alternative corrective measures to determine if a cost efficient, implementable solution exists which would prevent or reduce the storm damages.

A Section 103 study determines whether a cost effective solution exists which is in the Federal interest to implement. In order for a project to continue to design, at least one alternative plan investigated must have a benefit-to-cost ratio of 1.0 or greater. In addition, a local sponsor must be identified who is willing to cost share in the development of plans and specifications and in the implementation costs of the identified project.

West Silver Sands Beach is located along the southern shore of Connecticut on Long Island Sound in East Haven, Connecticut. The study area, which contains approximately 50 year-round homes and summer cottages, is bounded on the east by Caroline Creek, a tidal creek, on the north by a tidal marsh and on the south by Long Island Sound. A revetment is located along the western shore of Caroline Creek and continues seaward beyond the beach to form a terminal groin. To the north of the inland limit of the revetment, the shoreline is low as it forms the tidal marsh.

The problem at West Silver Sands Beach is flooding of the area from storm surge overtopping the banks of Caroline Creek and West Silver Sands Beach during minor storms, as well as flooding from high water levels within the tidal marsh. The tidal surge within the creek overtops the creek bank where the revetment ends and the tidal marsh begins. The continued erosion of the beach allows waves to overtop the beach, contributing to the backshore flooding. In addition, the storms cause elevated water levels in the tidal marsh to the north of the study area to inundate the primary access road to the area.

Residents of the area indicated that overtopping of the beach was not a major problem. However, during the investigation, hydraulic analyses indicated that Caroline Creek will overtop its bank during a storm with a one-year frequency, and the beach can be overtopped by waves during a storm with a two-year frequency. For this reason, an analysis of a beach nourishment alternative was required.

During the investigation, five alternative plans were formulated to reduce damages in the West Silver Sands Beach area. These included raising of residential structures and streets, installing a tide gate in Caroline Creek, beach nourishment, an offshore breakwater, and extension of the Caroline Creek revetment. These alternatives, or a combination of them, were all investigated as potential solutions to prevent flooding problems in the study area.

Based upon the analysis described in this report, none of the alternative plans analyzed resulted in a benefit-to-cost ratio of 1.0 or greater. Therefore, no Federal involvement is warranted and the West Silver Sands Beach Study is being terminated.

WEST SILVER SANDS BEACH, EAST HAVEN, CT
SHORE PROTECTION AND STORM DAMAGE REDUCTION STUDY
FEASIBILITY REPORT

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

WEST SILVER SANDS BEACH, EAST HAVEN, CT
SHORE PROTECTION AND STORM DAMAGE REDUCTION STUDY

FEASIBILITY REPORT

INTRODUCTION

This report was prepared by the New England Division, Corps of Engineers, in response to a May 23, 1994 request from the Town of East Haven, Connecticut. The Town's request came as a result of concerns about flooding along Brazos and Ellis Roads in the West Silver Sands Beach area, as well as potential damages resulting from the loss of the beach fronting the area.

West Silver Sands Beach is a 2,950 foot long beach located on the southern shore of Connecticut on Long Island Sound in the Town of East Haven, Connecticut. Approximately 1,200 feet of the beach was identified as the study area for this investigation. The limits of the study area were determined based upon the extent of existing flooding which occurs within the area. The Town of East Haven is located east of New Haven and approximately 40 miles south of Hartford. (See Vicinity Map, Figure 1.)

The area contains approximately 45 homes and summer cottages as well as roadways which have experienced flooding problems in the past during even minor storm events. Erosion has reduced the width of the beach to the extent that storm waters now overtop the beach causing additional flooding of the study area. The residents also have safety concerns since elevated water levels in the backshore tidal marsh cause a section of Brazos Street, the main access to this area, to flood during very small frequency storms, which they fear may result in the area becoming completely inaccessible or isolated.

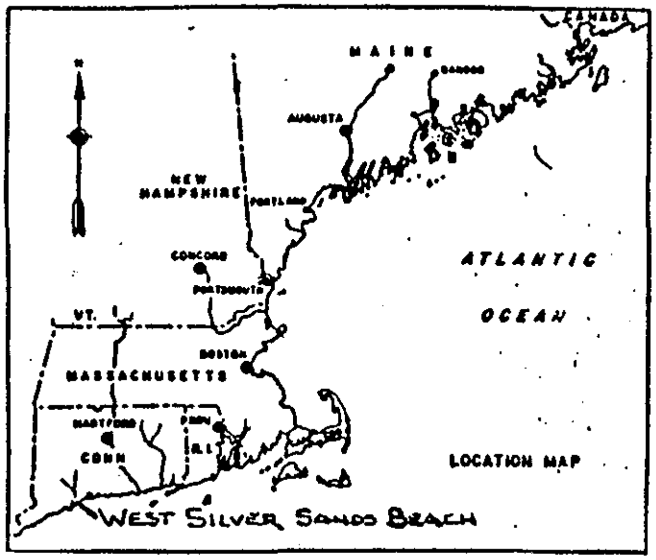
STUDY AUTHORITY

This report was prepared under the authority of Section 103 of the River and Harbor Act of 1962, (PL 87-874) as amended.

Section 103 authorizes the study, design and construction of civil works for the purpose of preventing shoreline flood and coastal damages resulting from storms.

STUDY PURPOSE AND SCOPE

The purpose of this study was to determine if cost effective solutions could be identified which would alleviate the storm damage problems being experienced in the West Silver Sands beach area of the town of East Haven, Connecticut. In addition to identifying cost efficient solutions, the study would also determine if a local sponsor was available and willing to cost share in the design and implementation of an economically justified project.



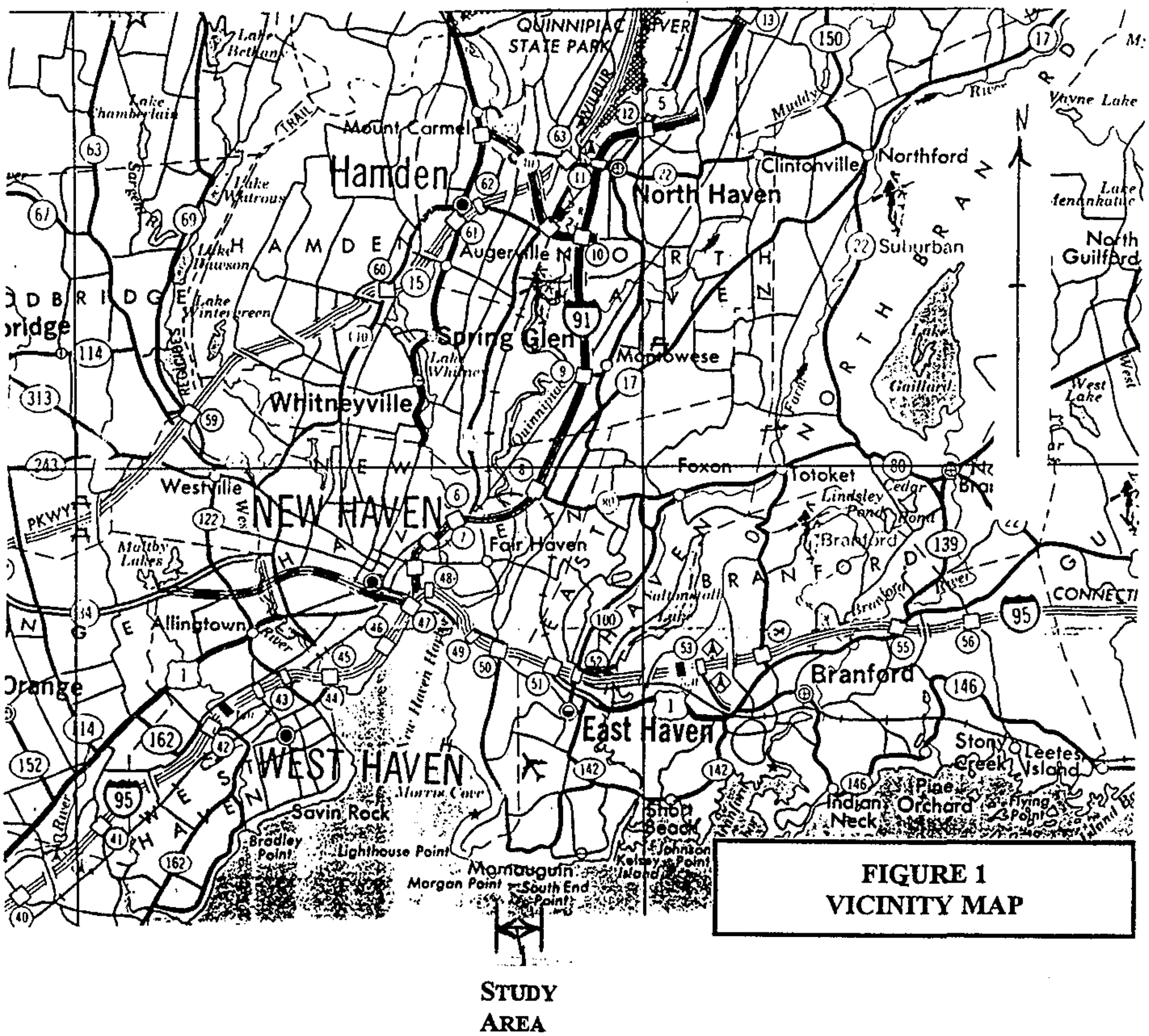


FIGURE 1
VICINITY MAP

PRIOR STUDIES AND IMPROVEMENTS

The Corps of Engineers has undertaken two previous studies within the area in question, although not specifically to investigate problems in the West Silver Sands Beach area.

The area was included in one of a series of cooperative studies undertaken with the State of Connecticut. The study which covered this area is entitled "Beach Erosion Control Report on Cooperative Study of Connecticut Area 9, East River to New Haven Harbor", dated August 12, 1955.

In addition, this area was included in a comprehensive Water Resources Study of Long Island Sound. A Tidal - Flood Management Reconnaissance Study of West Central Connecticut, completed in June 1988 discussed this area, however, none of the alternatives investigated at that time were found to be economically justified.

PLAN FORMULATION

EXISTING CONDITIONS

West Silver Sands Beach is located along the southern shore of Connecticut on Long Island Sound in East Haven, Connecticut. It is bounded on the east by Caroline Creek, a tidal creek; on the north by a tidal marsh and on the south by Long Island Sound. The southern portion of the western bank of Caroline Creek contains a rock revetment which continues seaward to form a terminal groin.

The study area covers approximately 1,200 feet of beach. The western limit of the study area was selected based on the extent of potential flooding determined by discussions with residents and an analysis of storm water overtopping rates for the area. (See Location Map, Figure 2.) The area contains approximately 45 year round homes and summer cottages. The beach within the study area is privately owned.

PROBLEMS AND OPPORTUNITIES

The problem at West Silver Sands Beach is backshore flooding as a result of storm waters overtopping the banks of Caroline Creek and West Silver Sands Beach. In addition, elevated water levels within the tidal marsh cause the inundation of a section of Brazos Street. Brazos Street is the main access to the study area. The residents are concerned that this flooding may make the road inaccessible to emergency vehicles and cause the area to become isolated, hampering evacuation efforts which may be necessary.

WITHOUT PROJECT CONDITION

The without project condition represents what will likely happen in the future if no Federal project is implemented to meet the areas needs. In the case of West Silver Sands Beach, it is assumed that the backshore flooding and erosion of the beach would continue as it has in the past. With the loss of additional beach area, it is likely that more storm waters will overtop the beach resulting in additional flooding of the backshore area. For the without project condition, damages currently experienced were assumed to continue to occur.

OBJECTIVES AND CONSTRAINTS

Planning objectives are identified to meet the problems and opportunities of the study area. The main planning objective for this shore protection project is to prevent further flooding damage in the West Silver Sands Beach area and to maintain access to the area during storms.

Planning constraints are natural, structural or institutional conditions which impose restrictions on the range of solutions available to meet the planning objectives described above. The Connecticut DEP is concerned that structural alternatives may create problems for this and other areas along the coastline. These problems could be such things as trapping natural

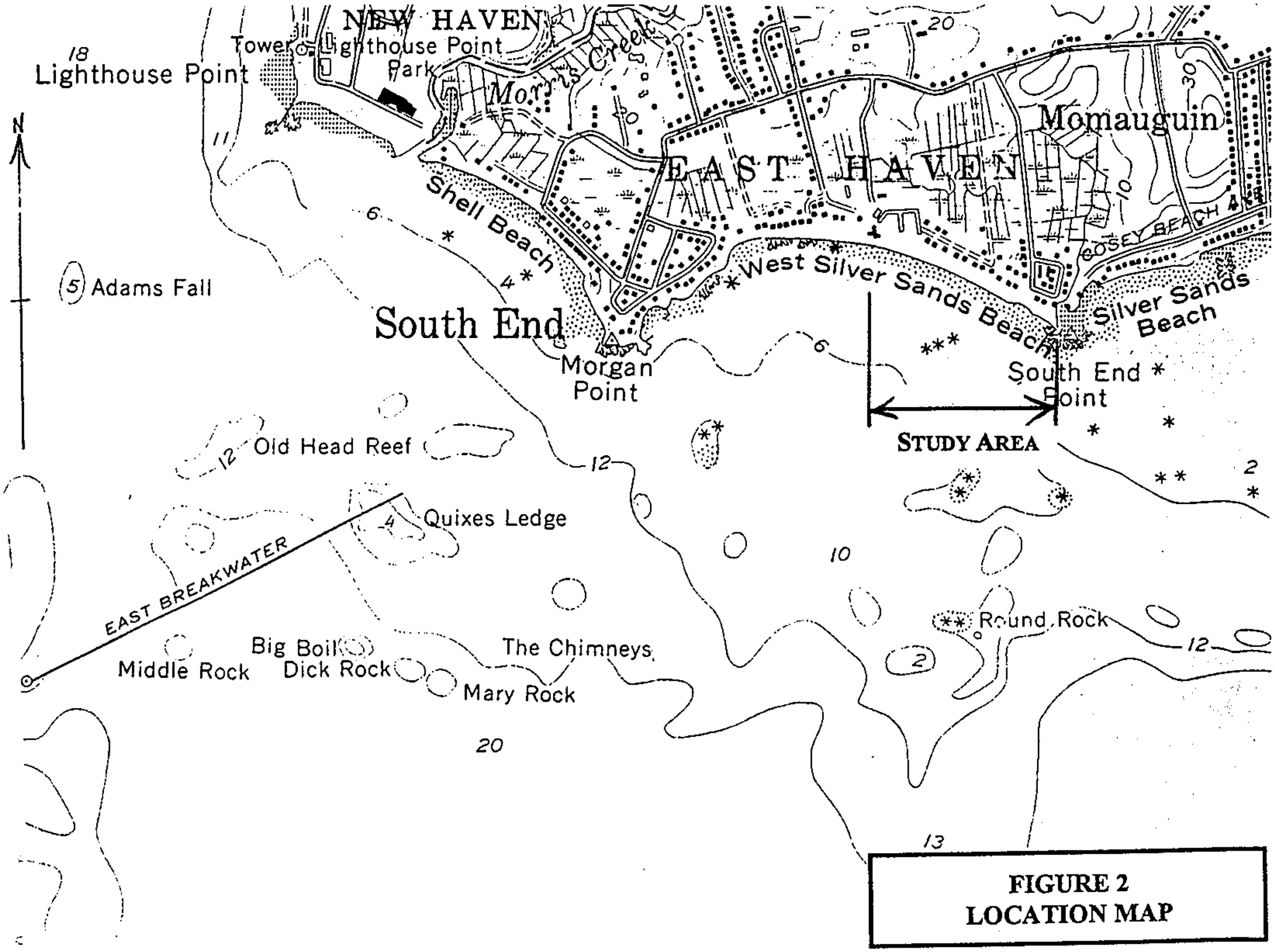


FIGURE 2
LOCATION MAP

nourishment materials thereby causing erosion of downdrift beaches, and scouring at the toes of structures which could increase the erosion rate near the structure. In order to present a full array of alternatives, structural measures are presented in this report. However, it should be recognized that the State of Connecticut is not typically supportive of such plans.

ALTERNATIVE PLANS

The following measures, alone or in combination were formulated for evaluation:

A. Raising residential structures and streets - This alternative would elevate structures and sections of streets in the area to eliminate the majority of the detrimental effects of flooding. This alternative would not, however, provide any mechanism to prevent the overtopping which causes the flooding to occur or the erosion of the beach. It would also not eliminate any minor residual damage to the exterior of the structure and grounds which may occur as a result of wind and wave action during extreme storms.

B. Construction of a tidal gate within Caroline Creek - The purpose of this alternative is to prevent tidal surges in Caroline Creek which currently overtop the banks, contributing to the flooding of the study area and the access road to the area. This alternative alone would not prevent flooding caused by storm waves overtopping the beach or those from backshore runoff. It would also not prevent further erosion of the beach.

C. Beach nourishment - This alternative includes the restoration of the beach which would eliminate storm wave overtopping which contributes to the flooding of the study area. Various levels of protection were analyzed for this alternative. Only a 100-year beach would prevent damages from storm waves impacting residential structures during severe storms. This alternative alone would not prevent flooding of the area caused by tidal surges in Caroline Creek or those from backshore runoff.

D. Construction of an offshore breakwater - The construction of an offshore breakwater of sufficient length and height to provide protection to the study area was investigated. The purpose of this alternative is to force the storm waves to break prior to reaching the beach area thereby minimizing wave runup and overtopping, resulting in a reduction of the erosion rate of the beach.

E. Extension of revetment on west bank of Caroline Creek - This alternative would provide for the continuation, to the north, of the existing revetment along the west bank of Caroline Creek in order to prevent the overtopping which occurs at the point at which the revetment currently ends and the low elevation of the tidal marsh begins. To provide more complete protection to this area, a portion of Brazos Road would also need to be raised. This alternative alone would not prevent flooding associated with storm waves overtopping the beach or backshore flooding. It would also not provide protection to the beach itself.

Combinations of the above described alternatives were also analyzed to determine which combinations of plans would be required to provide the most protection to the area in question.

None of the alternatives formulated would completely prevent flooding of the streets, since some of the street flooding results from backshore runoff combined with culvert restrictions and low lying roadways. Since most of the benefits to be gained in the study area are from prevention of damages to the structures, only those damages were analyzed. If the benefits generated from these alternatives had been sufficient, more detailed analysis would have been performed which would include alternative plans to resolve the low lying roadway flooding.

EVALUATION OF ALTERNATIVE PLANS

Alternative plans were evaluated on the basis of engineering design, estimated costs for plan implementation and maintenance, and the flood damage reduction and shore protection benefits to be derived from each plan. An alternative is economically justified if the annual benefits equal or outweigh the annual costs. Further Federal involvement can only be recommended if at least one alternative plan investigated results in a benefit-to-cost ratio of 1.0 or greater. The following sections describe the processes used for each element involved in the evaluation of plans. Details of the evaluations are contained in the appropriate appendices.

ENGINEERING ANALYSIS

An engineering analysis, including geotechnical input, was performed for the structural raising of houses, beach nourishment and the revetment extension plans. Design wave heights, wave runup and overtopping calculations were computed for existing and proposed conditions. The geotechnical analysis included a review of the existing surface conditions, materials availability for the alternatives, and design considerations for all alternatives. (Details of these analyses and cost estimates can be found in Appendices A, B, C and D.) The tide gate and breakwater plans were eliminated early in the analysis based upon the limitations of the plans, their negative environmental effects and their high costs.

The tide gate alternative would only prevent storm surges from entering the Caroline Creek area through the mouth of the creek. This alternative would not prevent storm waves from overtopping the beach fronting the study area. It would also have no effect on backshore runoff. In addition, there are many questions associated with this alternative regarding whether it would impact flushing of the backshore tidal marsh. For these reasons, this alternative was not evaluated in detail.

The breakwater alternative would only reduce storm waves directly impacting the area. It would not have an effect on tidal surges in the creek area or flooding caused by these surges. In addition, there are questions as to the potentially high costs normally associated with this alternative and the environmental impacts resulting from its construction. For these reasons, this alternative was not evaluated in detail.

Each remaining alternative, or combination of alternatives, was designed to eliminate the flooding occurring in the area as a result of storms, with the exception of some roadway flooding. Raising structures would greatly reduce the susceptibility of the structures to flood damage, but would not eliminate the flooding itself. The beachfill alternative would result in a large beach which would prevent storm waves and runup from reaching the backshore area, thereby eliminating the flooding potential from that source. The revetment extension and elevation of a small section of Brazos Road would prevent storm surges from overtopping the banks of Caroline Creek and flooding the study area. The beachfill and revetment extension plans in combination would eliminate the majority of the flooding occurring in the area, but they would not prevent the flooding which occurs as a result of backshore runoff. If sufficient benefits were generated from these alternatives, additional detailed analysis would have been undertaken which would have taken into consideration the low lying roadway flooding.

It is apparent that the only plan which would, by itself, significantly reduce damages to the structures associated with flooding would be house raising. House raising would not, however, prevent residual damages to the outside of the structure or grounds which may occur from storm wave damages during severe storms. It would also not prevent flooding of the low lying roadway area which, according to the residents, may cause the area to become inaccessible and hamper any evacuation efforts which may be required during major storms. The hydrology and hydraulic analysis did review flooding of the streets. Due to the fact that the majority of the benefits would be from the reduction of damages to the structures, the prevention of roadway flooding and residual wave damage was not pursued.

The analyses of the alternative plans were based upon design wave heights, runup and overtopping for storms with return periods of 10, 50 and 100 years. Preliminary total cost estimates and annual costs for the alternative plans evaluated are illustrated in Table 1.

TABLE 1

TOTAL COSTS AND ANNUAL COSTS FOR ALTERNATIVE PLANS
(Based upon 6/95 price levels and an interest rate of 7-5/8% over 50 years)

PLAN	DESCRIPTION	TOTAL COST	ANNUAL COSTS
A	Raise houses	$ 1,810,700	$ 141,700
B	Tidal Gate	**No Cost Analysis Performed**	
C	Beach Nourishment		
	10-yr (1,200 ft)	$ 2,358,300	$ 184,500
	50-yr (1,200 ft)	$ 3,243,300	$ 253,700
	100-yr (1,200 ft)	$ 4,544,600	$ 355,500
D	Offshore Breakwater	**No Cost Analysis Performed**	
E	Extension of revetment	$1,324,600	$ 103,600
C + E	Beach and revetment		
	10-yr (1,200 ft)	$ 3,682,900	$ 288,100
	50-yr (1,200 ft)	$ 4,567,900	$ 357,300
	100-yr (1,200 ft)	$ 5,869,200	$ 459,100

ECONOMIC ANALYSIS

The purpose of the economic analysis is to determine if alternative plans are economically justified. Appendix E, Economic Analysis, presents details of the benefits of the various alternatives. The benefits are determined based upon a comparison of the without project condition to the with project condition. For the purpose of this analysis, it was projected that without a Federal project the damages being experienced in the area would continue. As was explained above, in order to prevent flooding damage, the beach and revetment extension would have to be combined. The house raising alternative would stand alone. However, with the exception of the 100-year beach alternative plan, there would be some residual damages from wind and wave impact to the exterior of structures and grounds during extreme storms. Table 2 identifies annual benefits for the three beach levels analyzed (10, 50, and 100-years) in conjunction with the revetment extension, as well as the house raising. Table 3 presents a comparison of benefits and costs for the alternative plans.

TABLE 2

BENEFITS FOR ALTERNATIVE PLANS

ALTERNATIVE	ANNUAL BENEFITS
10-year Beach with Revetment Extension	$ 31,750
50-year Beach with Revetment Extension	$ 40,780
100-year Beach with Revetment Extension	$ 40,810
House Raising (to above 100-yr level)	$ 35,740

TABLE 3

COMPARISON OF ALTERNATIVE PLANS

ALTERNATIVE	ANNUAL BENEFITS	ANNUAL COSTS	BENEFIT TO COST RATIO	NET ANNUAL BENEFITS
10-yr beach with revetment extension	$ 31,750	$ 288,100	0.11	none
50-yr beach with revetment extension	$ 40,780	$ 357,300	0.11	none
100-yr beach with revetment extension	$ 40,810	$ 459,100	0.09	none
House Raising (above 100-yr level)	$ 35,740	$ 141,700	0.25	none

CONCLUSIONS

The New England Division, Corps of Engineers, has reviewed and evaluated all pertinent data concerning the proposed plans of improvement. The views of concerned interests were considered, relative to the storm damage problems occurring in the area. .The study area experiences flooding during even minor storm events and these problems are projected to continue if nothing is done to correct the problem. The flooding comes from three different areas: waves overtopping the beach, tidal surges overtopping the western bank of Caroline Creek, and elevated water levels in the tidal marsh area behind the study area. In addition, if the erosion of the beach is allowed to continue, the severity of the flooding will likely increase.

However, based upon the analysis described in this report, it was found that the economic benefits identified were not sufficient to justify the costs of the proposed improvements. This investigation of storm damage reduction of West Silver Sands Beach, therefore, cannot recommend future Federal involvement in implementation of improvements.

RECOMMENDATIONS

I have concluded that no further study of storm damage reduction improvements for West Silver Sands Beach, East Haven, Connecticut, is warranted at this time and have terminated further investigation.

Earle C. Richardson
Colonel, Corps of Engineers
Division Engineer

APPENDICES

APPENDIX A
COASTAL ENGINEERING ANALYSIS

WEST SILVER SANDS BEACH
FLOOD DAMAGE REDUCTION
FEASIBILITY STUDY
EAST HAVEN, CONNECTICUT

APPENDIX A
COASTAL ENGINEERING

Prepared By:
Coastal Engineering and
Survey Branch
Design Division
Engineering Directorate

Department of the Army
Corps of Engineers
New England Division

December 1995

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. GENERAL DESIGN CONSIDERATIONS

1.1 Study Area and Reaches:

The study area (see location map, FIGURE A-1) is located along the north shore of Long Island Sound and involves the coastal area between Morgan Point and South End Point in the Momauguin area of East Haven, Connecticut. There is about 3/4 of a mile of coastline between these two points and about 5/8 of a mile of beach involved in this study. After reviewing the coastal maps for the study area, it was determined that the subject beach was exposed to storms out of the southwest, south, and southeast.

2. DESIGN PARAMETERS

2.1 Winds:

Windspeeds for this study were obtained from Frequency of Adjusted Annual Maximum Windspeed (MPH) Stratford, Connecticut. This data, with some adjustment to account for local anomalies, was used to develop the wave climate and top of runup in the study area. The adjusted windspeeds used in this study are shown on Table A-1 of this appendix.

2.2 Duration and Fetch Limited Waves:

In all water basins the response to a storm is to try to reach a fully developed sea (also referred to as a fully arisen sea). For such a sea, it can be stated that winds and waves are in equilibrium and because of the physical limitations of water characteristics, further wave development is not possible. In a fully arisen sea, as the storm continues, any additional wave growth ultimately causes the wave to exceed its maximum sustainable steepness and the wave collapses limiting further development. Although a fully developed sea is the ultimate consequence of storm-induced wave growth, factors may keep this state from ever being reached. If the storm duration is limited, the wind eventually subsides and the larger waves decay leaving only the shorter period waves. This results in a duration limited wave sea condition. Also, the physical conditions of the basin may so restrict the amount of time the wind and water can interact before obstructions disturb this interaction that larger waves can never develop. Such a situation results in the fetch limited wave and regardless of how long the storm continues, no larger wave growth will occur.

2.3 Duration:

Storm durations where taken to be 1, 3, and 8 hours. In that way, the adjusted average windspeeds are applied directly, and the resulting wave heights could then be attributed to the event frequency represented by that event duration and its associated windspeed.

2.4 Fetch:

Fetch distance is also critical for the development of the wave field. The unobstructed distance along the compass directions of interest (fetch) were scaled from existing coastal maps of Long Island Sound, and were converted into statute miles. The fetch distances used in the development of the wave climate for each fetch were determined to be 32, 20, and 24 miles, for winds out of the southwest, south, and southeast, respectively.

2.5 Wave Climate:

In this study, all waves were allowed to develop in deep water and thereby become as fully developed as possible. In Table A-2, the waves and wave periods are shown along with the limiting factors (D, duration or F, fetch) affecting wave growth for the three wind directions and five event frequencies involved in this study. For the purpose of designing coastal structures, the maximum wave group shown for each fetch was used.

2.6 Wave Refraction:

For purposes of this study, on shore wave directions including those from a direction other than normal to the shore are considered. This is due to the effect of wave refraction. Deep water waves are influenced by the coastal bathymetry and bend towards the coastline approaching a direction normal to the shore as they move shoreward.

2.7 Water Depth:

As the deepwater waves approach the shore, they come under the influence of coastal bathymetry and as the water depth becomes less, bottom friction helps to reduce the height of the wave. Current theory states that the maximum stable wave height be taken as 78% of the depth of the water. The problem then becomes one of establishing a water depth along a coastline. From this depth, a design wave can be established. This depth can be described as the still water level (SWL).

The SWL can be made up of the tide level, storm surge, and wind setup. For coastal designs, wave setup (Sw) is an additional factor that may need to be considered and added to obtain water depth. Wave setup is a local phenomena resulting from wave action alone resulting in a "superelevation of the mean water level". It was assumed that Sw was the result of a monochromatic wave train and sufficient time exists to establish an equilibrium water level. Table A-3 provides the SWL used to evaluate proposed coastal structures.

3. PROTECTIVE STRUCTURES

3.1 Sandfill:

When waves impact on the shore or on a structure, the energy within the wave is dissipated in part by running up on the shore or structure. This runup height (Ru) is measured from the initial SWL. Therefore, the height of a structure needed to prevent overtopping is the sum of the Ru plus the SWL.

The beach berm elevation needed to prevent overtopping was established by determining the Ru and adding it to the SWL previously determined. The runup height was determined using the U.S. Army Corps of Engineers' computer program, ACES 107e, Irregular Wave Runup on Beaches, where the applied wave height was the smaller of either the deep water wave or 78% of the SWL; and the beach slope was allowed to be 1 foot vertical to 15 feet horizontal. Table A-4, in Section 3.2, shows the berm elevations determined for each event frequency studied. Because of the conservative allowances made establishing the SWL, no freeboard height was added.

3.2 Design Sections and Quantities:

Existing beach cross section data was obtained from topographic maps of the West Silver Sands Beach area provided by the town. Sixteen cross sections were generated which represent the beach from South End Point to higher ground along Morgan Avenue at the western side of the beach. The map contour data was adjusted from mean sea level (msl) to National Geodetic Vertical Datum (NGVD) by adding 0.8 ft and the distances between contours were scaled from the map. The offshore sounding data and offshore slopes used were determined from various coastal maps for this area.

It was determined that a sandfill profile (FIGURE A-2) with a 50 ft. wide berm with a seaward face sloping from the berm to natural ground, at a rate of 1 ft. vertical to 15 ft. horizontal, and a landward face rising from natural ground to the berm, at a rate of 1 ft. vertical to 5 ft. horizontal, would be the most suitable for a sandfill profile in this area. The berm top elevation was determined from the runup calculations in 3.1 above and superimposing this profile on the beach cross sections the volume of sandfill was determined.

Two schemes were requested to be evaluated for this study to protect this beach area. Both start from South End Point and continue westward. The two vary with respect to the extent of protection they provide to the backshore properties . The first scheme provides coverage over 3,000± ft. of beach (full coverage), while the second scheme restricts coverage to the first 1,650± ft. of beach (limited coverage). The 10 yr., the 50 yr., and the 100 yr. storm events, as requested, were investigated. The volume of sand needed and the top berm elevations required to protect against overtopping due to direct wave attack was determined for these storm events and protection schemes. The conclusions of those investigations are as follows:

TABLE A-4

Coverage:		Limited 1,650± Ft.	Full 3,000± Ft.
Storm Event (Yrs.)	Berm El. above NGVD (Ft.)	Sand Volume (CY)	Sand Volume (CY)
10	12.0	90,000	189,000
50	13.5	124,000	257,000
100	15.5	174,000	355,000

4. <u>CONCLUSIONS</u>

4.1. Protection provided:

As stated, both schemes will provide protection within the limit of their length along the beach against overtopping due to direct storm attack. However, neither, by themselves, will prevent flooding to the backshore. Adjacent shore areas have elevations which are lower than storm surge water elevations associated with the storm events included in this study. Accordingly, if the protection is not extended through these locations flooding will continue in the backshore.

5. <u>RECOMMENDATIONS</u>

5.1 Additional Studies:

The wave climates and volume were determined to satisfy the general conditions of a feasibility level study of this area. If further study is warranted, the specific needs of a site must be considered and if necessary reinvestigated for wave heights and sandfill volumes required. This reinvestigation would take into account any existing unique design conditions. Also, prior to concluding that a beach berm may be suitable at a specific site, the littoral transport occurring along that coastline must be determined. This work may include both cross shore and longshore sand transport effects. Without such an investigation it would be impossible to state with any assurance that the coastal protection desired would be provided over the expected life of the structure.

6. REFERENCES

EM 1110-2-3301, 31 May 1995, Design of Beach Fills, U.S. Army Corps of Engineers

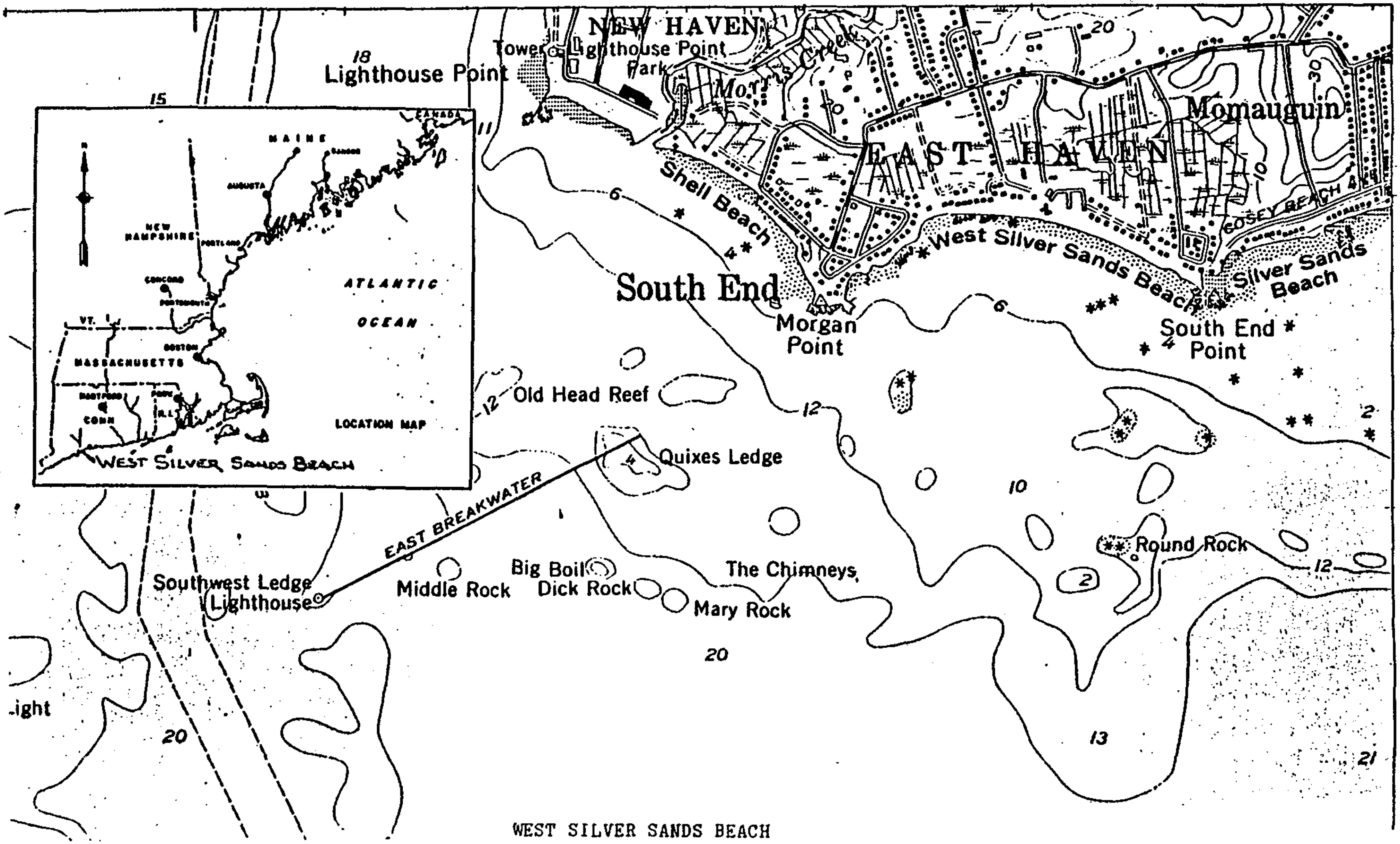
EM 1110-2-1412, 15 April 1986, Engineering and Design Storm Surge Analysis, U.S. Army Corps of Engineers

EM 1110-2-1414, 7 July 89, Water Levels and Wave Heights For Coastal Engineering Design, U.S. Army Corps of Engineers

EM 1110-2-1502, 20 August 1992, Coastal Littoral Transport, U.S. Army Corps of Engineers

Tidal Flood Profiles New England Coastline, September 1988, U.S. Army Corps of Engineers, New England Division

Shore Protection Manual, 2 Vol., 4th Ed., 1984, U.S. Army Corps of Engineers



WEST SILVER SANDS BEACH
EAST HAVEN, CT

FIGURE 1

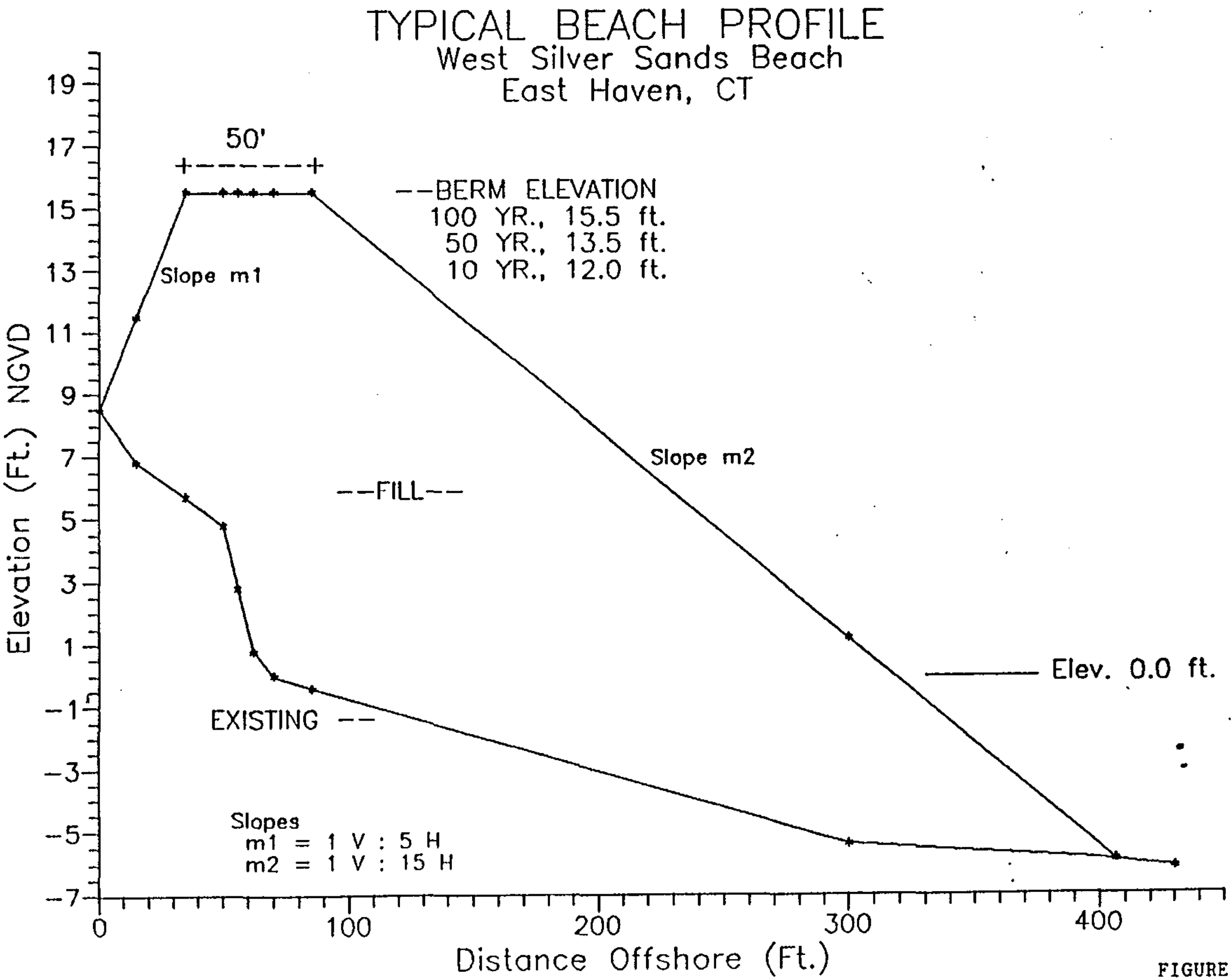

TYPICAL BEACH PROFILE
West Silver Sands Beach
East Haven, CT
50'
--BERM ELEVATION
100 YR., 15.5 ft.
50 YR., 13.5 ft.
10 YR., 12.0 ft.
Slope m1
Slope m2
--FILL--
Elev. 0.0 ft.
EXISTING --
Slopes
m1 = 1 V : 5 H
m2 = 1 V : 15 H
Elevation (Ft.) NGVD
Distance Offshore (Ft.)

FIGURE 2

West Silver Sands, East Haven, CT,

TABLE 1

AVERAGE WIND SPEEDS (MPH)
ADJUSTED TO PRODUCE CONSISTENT WIND STRESS

DIRECTION	DURATION HR.	EVENT FREQUENCY YEARS 2	5	10	50	100
SE	1	22	29	33	44	50
	3	18	21	22	30	35
	8	16	19	21	28	33
S	1	28	33	35	41	44
	3	25	28	30	34	36
	8	22	26	28	32	34
SW	1	26	30	32	39	43
	3	22	26	30	36	38
	8	20	23	24	29	31

West Silver Sands, East Haven, CT,

TABLE 2

WAVE HEIGHT:PERIOD (FT.: SEC.)

DIRECTION	AVG FETCH (MI)	DURATION HR.	EVENT FREQUENCY YEARS 2		5		10		50		100	
SE	24.0	1	1.2 :2.3	D	1.9 :2.8	D	2.3 :3.0	D	3.7 :3.8	D	4.6 :4.2	D
		3	1.9 :3.1	D	2.4 :3.4	D	2.6 :3.5	D	4.3 :4.4	D	5.5 :5.0	D
		8	2.6 :4.2	F	3.2 :4.5	F	3.6 :4.7	F	5.2 :5.3	F	6.4 :5.7	F
S	20.0	1	1.8 :2.7	D	2.3 :3.0	D	2.5 :3.2	D	3.3 :3.6	D	3.7 :3.8	D
		3	3.2 :3.9	D	3.8 :4.2	D	4.3 :4.4	D	5.3 :4.9	D	5.8 :5.1	D
		8	3.5 :4.5	F	4.3 :4.8	F	4.7 :5.0	F	5.6 :5.3	F	6.1 :5.4	F
SW	32.0	1	1.6 :2.5	D	2.0 :2.8	D	2.2 :3.0	D	3.0 :3.4	D	3.6 :3.7	D
		3	2.6 :3.5	D	3.4 :4.0	D	4.3 :4.5	D	5.8 :5.1	D	6.3 :5.3	D
		8	3.9 :5.0	F	4.7 :5.4	F	4.9 :5.4	F	6.3 :5.9	F	6.8 :6.1	F

D= Duration Limited Wave
F= Fetch Limited Wave

TABLE 3

STILL WATER LEVELS
SWL (MLW)

		EVENT FREQUENCY YEARS 2	5	10	50	100
SE	STORM TIDE	9.6	10.4	10.9	12.4	13.0
	WAVE SET-UP	0.1	0.2	0.2	0.3	0.4
	SWL	9.7	10.6	11.1	12.7	13.4
S	STORM TIDE	9.6	10.4	10.9	12.4	13.0
	WAVE SET-UP	0.2	0.2	0.3	0.3	0.4
	SWL	9.8	10.6	11.2	12.7	13.4
SW	STORM TIDE	9.6	10.4	10.9	12.4	13.0
	WAVE SET-UP	0.2	0.2	0.3	0.3	0.4
	SWL	9.8	10.6	11.2	12.7	13.4

AVERAGE SWL	2	5	10	50	100
MLW	9.8	10.6	11.2	12.7	13.4
NGVD	7.4	8.2	8.8	10.3	11.0
MSL	6.6	7.4	8.0	9.5	10.2

SWL = STORM TIDE + WAVE SET-UP
0.0 NGVD = -0.8 MSL = +2.4 MLW

APPENDIX B
GEOTECHNICAL ANALYSIS

SECTION 103 FEASIBILITY STUDY
WEST SILVER SANDS BEACH
EAST HAVEN, CONNECTICUT

GEOTECHNICAL
APPENDIX B

April 1996

SECTION 103 FEASIBILITY STUDY
WEST SILVER SANDS BEACH
EAST HAVEN, CONNECTICUT
GEOTECHNICAL APPENDIX B

FIGURES | Figure No.

SECTION 103 FEASIBILITY STUDY
WEST SILVER SANDS BEACH
EAST HAVEN, CONNECTICUT
GEOTECHNICAL APPENDIX B

1. INTRODUCTION

The West Silver Sands Beach study area is located along the Connecticut coast, east of New Haven Harbor, and is bounded by Caroline Creek to the east, a low-lying marsh to the north, and the beach front extending west from South End Point (see Figure B-1). The study area encompasses a small residential community consisting of approximately 50 homes.

The proposed alternatives considered for reducing flood damages to the study area consist of house raising, a tidal gate on Caroline Creek, beach nourishment, an offshore breakwater, extending the revetment west of Caroline Creek, and various combinations of these features. None of the alternatives standing alone are capable of reducing the damages from both storm wave overtopping and backshore flooding. The combination of alternatives considered most effective in reducing damages consists of constructing a concrete I-wall between the residential area and the creek and marsh to the east and north; raising Brazos Road onto a newly constructed dike; and raising the beachfront by adding sand. Two scenarios are considered for beach nourishment. Limited coverage would include only the first 1,650 ft of beach west of South End Point. Full coverage would include approximately 3,000 ft of beach.

The concrete I-Wall and dike combination will reduce the backshore flooding from Caroline Creek. The concrete I-Wall will start at South End Point and run along the west bank of Caroline Creek for approximately 700 feet and then turn northwest towards Brazos Road where it will continue for approximately 430 feet into the dike (Figure B-5). Brazos Road will be raised onto the dike. This will help to eliminate the amount of construction in the marsh and provide access during high water. Presently, there is a concrete block wall with dumped armor stone in front of it along the west bank of the mouth of Caroline Creek.

2. TOPOGRAPHY

The land immediately backing this section of the coast is a low-lying surface having little relief, with most elevations less than 10 ft NGVD, except where glacial till was deposited, resulting in elevations of 10 to 40 ft NGVD.

Topography in the study area has been influenced by four factors: presence of bedrock at or near the surface, the effects of glaciation, coastal processes, and construction by man.

3. GEOLOGY

3.1 General. The geology of New England is the result of a complicated history of orogeny, intrusion, and metamorphism. There are mixed rock types in very complex associations. Although numerous faults have been mapped or otherwise suspected, none are presently known to be active. The area has been glaciated several times and the modern landscape is largely one of thin, remnant surficial deposits of glacial origin overlying bedrock. The retreat of the glaciers brought about a rise in sea level, accompanied by the rebound of the land as it was unloaded. This trend appears to have stabilized, and now the New England region appears to be subsiding very slowly. Sea level continues to rise gradually. The current trend of sea level rise, measured along the Connecticut coast, is approximately 0.1 ft per decade (Department of Commerce, National Oceanic and Atmospheric Administration, 1988).

3.2 Bedrock Geology. The study area is underlain by the Light House Gneiss, a metamorphosed igneous rock formation of Proterozoic age (Rodgers, 1985) (see Figure B-2). Contouring of the bedrock surface shows bedrock to be very shallow in the study area (at or slightly above mean sea level) (Brown, 1974b). Geologic maps of the study area show numerous, small bedrock outcrops in the area behind (inland of) West Silver Sands Beach (Brown, 1974a). South End Point and Morgan Point are both prominent bedrock headlands (see Figure B-3). The Light House Gneiss is described as a light pink or gray to red, medium-grained, well-foliated granitic gneiss (Rodgers, 1985).

3.3 Surficial Geology. Sediments overlying the bedrock are glacial till, stratified drift (sand and gravel outwash, with minor silt, clay, and till), marsh deposits, and beach deposits (see Figure B-4).

3.3.1 Glacial Geology. A thin layer of glacial till mantles the bedrock through most of the area backing the study area. Glacial till in this area is typically described as either a loose sand with numerous cobbles and boulders (supraglacial till) or a compact, poorly sorted deposit of silty, gravelly sand, with variable amounts of cobbles and boulders, and trace clay (lodgement till). Supraglacial till was deposited as the materials within the ice sheet melted out, and were let down on the existing landscape. Lodgement till was deposited at the base of the glacier, and, as a result, is very compact. The relatively loose, gravelly, cobbly sand (supraglacial till) is reported to be less than 10 ft thick on average (Brown, 1974a). Supraglacial till deposits (where present) overlie the lodgement till.

Marsh deposits are mapped north of the study area, and generally consist of variable mixtures of silt, sand, clay, and organic matter. These deposits are prone to being highly compressible in nature. The residential area and Brazos Road are shown to be built on artificial fill, which was likely placed over marsh deposits in order to provide adequate foundation conditions for the structures.

3.3.2 Coastal Geology. The specific nature and configuration of the shoreline in a given area are the result of a number of elements at work, on both small and large scales, within the complex beach environment. Such factors include the interaction of the tides and wave action with the situation of the existing shoreline, the resistance of the bordering mainland, and the dynamics of a slowly rising sea level.

In general, the coast betweeen Lighthouse Point (New Haven) and Guilford Point is dominated by exposed rocky headlands, separated by deep marshy coves, estuaries and a few small "pocket beaches" formed between rocky headlands (Bloom, 1967). West Silver Sands Beach is an example of a "pocket" beach, formed between the rocky headlands of South End Point and Morgan Point.

Beach materials observed during the site visit consisted of pinkish to reddish brown, subangular to subrounded, medium to coarse-grained sand, with little fine gravel (approximately 14%). A sample of the native beach was collected and subjected to mechanical analysis. The beach sand has a relatively high orthoclase feldspar content, resulting in the pinkish color. Small pebbles of red siltstone and sandstone are also present as a minor constituent. The glacial deposits are the primary parent material for the beach sands. The glacial deposits in this area were probably derived primarily from the local granitic bedrock, with a minor contribution from the Mesozoic metasedimentary and metavolcanic bedrock types north of the Triassic Border Fault. The mineralogy of the beach deposits reflect the glacial deposits' source area.

4. GENERALIZED SUBSURFACE CONDITIONS

Subsurface conditions were evaluated by reviewing geologic literature and making interpretations based on observations made during the site visit. There was limited subsurface information readily available from other projects in the study area. Soil explorations and laboratory testing were not performed, as they are not within the scope of a reconnaissance level study.

A generalized soil profile along the wall alignment appears to be variable thicknesses of artificial fill overlying compressible marsh deposits and granitic bedrock. Depths to bedrock are expected to be shallow (roughly 5 ft to 15 ft below ground surface), and to vary slightly across the study area. The subsurface profile along the beach likely consists of granular beach deposits overlying shallow bedrock.

5. DESIGN AND CONSTRUCTION CONSIDERATIONS

5.1 Dike under Brazos Road. Preliminary revetment sections were developed using the principles and procedures in the "Shore Protection Manual". The sections are shown on Figures B-6 and B-7. The revetment sections were used to calculate the quantities needed to prepare the estimates shown in the main report. Design wave heights and stillwater levels were provided by NED's Coastal Engineering and Survey Branch and Water Control Division and are more fully discussed in their appendices. Because the dike is located in the back shore, the top elevation of the impervious fill in the dike was the 100-year still water level (El. 10.7). Minimal wave heights were used to determine the size of armor stone for protection.

Formal stability, settlement and seepage analyses were not performed on the sections because there are no exploration or laboratory test data available for use in detailed analyses. Based on NED experience with similar materials the proposed 1 vertical on 1.5 horizontal slopes are judged to be safe. The proposed bedding layers have been designed as filters to prevent migration of fines.

5.2 Concrete I-Wall. Preliminary I-Wall sections were based on design guidance in EM 1110-2-2502 Retaining and Flood Walls. Typical sections are shown on Figures B-8 and B-9. The foot print of the constructed wall is less than 10 feet; whereas the foot print of the dike is approximately 70 feet. The I-Wall through the marsh area will reduce the amount of impact on the marsh environment. Design wave heights, stillwater levels, and top elevation of the wall were provided by NED's Coastal Engineering and Survey Branch and Water Control Division. Preliminary design of the I-Wall in the marsh area assumed there would be minimal wave action due to the backshore location. The seaside I-Wall was designed to withstand wave action. Detailed design analysis for the I-Wall will be performed at the feasibility stage if further study is warranted.

5.3 Constructability. Constructability and accessability to the site are good. Access to the area is by Brazos Road or Fairview Road. Appropriate permits will need to be approved prior to building within the marsh area. Some access across private property from Ellis Road will be necessary. The depth of bedrock will be a determining factor in the sheet pile design of the concrete I-Wall. Bedrock is expected to be relatively shallow at the site.

Sand may be delivered to the site by truck, or possibly by barge. All other materials for the dike and concrete wall would likely have to be trucked to the site.

6. MATERIAL AVAILABILITY

A materials availability survey was conducted for the materials required for the beachfill/concrete I-wall/dike alternative. Thus, materials for each were investigated.

6.1 Dike and Concrete I-Wall Materials.

6.1.1 General. Sources within a radius of approximately 40 to 50 miles of the study area were contacted to determine the availability of the materials required for this portion of the project.

6.1.2 Dike Materials (Figures B-6 and B-7).

6.1.2.1 Gravel Bedding. The material required is a bank run gravel, approximately 1" to 3" in diameter and graded (100% passing the 5" sieve). The quantity of gravel required for the project of a 100 year storm protection is approximately 3,000 cubic yards.

Four sources have been identified as being capable of producing the material in the quantity required. Two sources are located at distances of approximately 10 to 20 miles from the study area (North Haven and Beacon Falls). Two sources are located at distances of approximately 40 to 50 miles from the study area (Torrington and Lisbon).

6.1.2.2 Stone Bedding. The material required is a crushed stone, approximately 4" to 8" in diameter. The quantity of crushed stone (stone bedding) required for the 100 year project is approximately 1,900 tons.

Three sources have been identified as being capable of producing the material in the quantity required. These three sources are located at distances of approximately 10 to 20 miles from the study area (North Haven, North Branford, and Naugatuck). The North Branford source routinely produces a 3" to 9" "surge stone," which would still likely be acceptable for use on this project.

Three sources located at distances of approximately 30 to 40 miles from the study area were also contacted (Meriden, Woodbury, and Newington). The Meriden and Newington sources stated that they do not usually go to the coast, as it is too far for them to work competitively. The Woodbury source routinely produces a 6" to 10" diameter "modified rip rap," but commented that it was difficult to produce the 4" to 8" size stone at his plant. The modified rip rap would likely be slightly too large for the intended application.

6.1.2.3 Stone Protection. The stone required ranges between 180 and 400 lbs per stone for the armor stone. The quantity of armor stone required for the 100-year project is approximately 3,900 tons.

Six stone suppliers were contacted. Three sources located at distances of approximately 10 to 20 miles from the study area are capable of producing the stone required, but are not capable of delivering the stone to the site (North Branford, Stony Creek, and Naugatuck). One source, located at a distance of approximately 30 miles, indicated that it could produce and deliver the stone required (Woodbury). Two sources located at distances of 25 to 35 miles from the study area considered the site too far away for their prices to be competitive (Meriden and Newington).

6.1.2.4 Other material needed. Material required for the construction of the dike also includes 9,000 cubic yards of impervious fill, 2,500 cubic yards of pervious fill, 400 cubic yards of topsoil and seed, and 315 cubic yards of bituminous concrete.

Four sources of impervious fill were identified within 10 to 30 miles of the study area (North Haven, Meriden, Naugatuck, and Southbury). For this application, the specification for impervious fill has a fairly wide gradation envelope. As a result, a wide range of materials are available that would meet the specification. Cost varies considerably for these products, as some of them are specially processed for particular applications.

The sand required for beach fill would also be considered acceptable "pervious fill." Sources capable of producing beach fill are discussed in paragraph 6.2.3.

Topsoil, seed, and bituminous concrete are considered common building materials, as they are routinely produced and readily available in the vicinity of the study area.

6.1.3 Concrete I-Wall Materials (Figures B-8 and B-9).

The materials required for construction of a concrete I-Wall include concrete, PZ-22 sheet piles, granular fill and use of the existing stone for the toe in front of the I-wall.

6.1.3.1 Gravel. The gravel needed and sources available are the same as described in paragraph 6.1.2.1 above. Quantity required is approximately 550 cubic yards.

6.1.3.2 Existing Stone. The dumped armor stone in front of the existing block concrete wall will be reused for the concrete wall toe protection.

6.1.3.3 Other Material. Quantity of concrete needed is approximately 800 cubic yards. Impervious fill needed is approximately 400 cubic yards. Reference paragraph 6.1.2.4.

6.2 Beachfill Material.

The development of materials specifications for beachfill requires a comprehensive approach in characterizing the native beach materials, investigating the availability of borrow material sources, and evaluating the suitability and compatibility of these materials.

Quality criteria are stipulated in the beachfill specification to ensure that sound, durable, suitable materials of natural origin are provided. Specified criteria include minimum specific gravity (2.60), and particle shape (subangular to subrounded). The angular products of rock crushing operations are prohibited. Sand for beachfill must be free of contaminants, roots, organic matter, trash or other detritus, and must not contain more than 5 % friable particles, mica, or other flakes. Reddish sands, such as those derived primarily from the underlying sedimentary red beds of the Central Connecticut Lowland (west of the Triassic Border Fault), are not considered acceptable due to their tendency to weather and break down into finer material in the beach environment.

6.2.1 Methods of Grain Size Analysis. Two methods of grain size data analysis are used in the fields of coastal engineering and geology (sedimentology) to study sediment gradations. The Corps presents a method for evaluating the compatibility of proposed beachfill borrow materials, using the sedimentology approach to compare native and borrow material gradations, in EM 1110-2-3301. However, the Corps uses the engineering method to specify materials, since this is the same method used in the materials industry.

The engineering field and the material industry tend to work with grain size diameters (D), measured in millimeters (mm). Grain size analyses are conducted in accordance with ASTM (American Society for Testing Materials) Methods D 422 and D 2487. The results are plotted as gradation curves, in terms of "percent passing" ("percent finer than") a particular sieve. For example, the diameter which 84 percent of the sample is finer than is called the D84.

Generally, a borrow material that is well-gradede and coarser than the native beach material. The New England Division (NED) Corps specification envelopes for this project (shown in Figure B-10) are described with regards to various sieve sizes as follows:

U.S. Standard Sieve Size	% Passing
3/8	100
4	100 - 80
8	100 - 55
16	85 - 35
40	50 - 15
100	10 - 0
200	5 - 0

The envelope for the State of Connecticut Department of Transportation Specification for Fine Aggregate or "concrete sand" (M.03.01-2) is also plotted in Figure B-11, as it provides an indication of materials that are routinely produced and readily available commercially.

The field of sedimentology relies upon the same grain size testing methods, but uses the ϕ ("phi") scale in the analysis of grain size data, which is related to the grain size diameter, D (mm), as follows:

$$\phi = -\frac{\ln D}{\ln 2} \qquad D = \frac{1}{2^{\phi}} = 2^{-\phi}$$

Two parameters are of interest in this analysis: Phi Mean (M_ϕ) and Sigma Phi (S_ϕ). They are calculated as follows:

$$M_\phi = \frac{\phi 84 + \phi 16}{2} \quad \text{and} \quad S_\phi = \frac{\phi 84 - \phi 16}{2}$$

Because of the differences in the way that grain size data are plotted in the different systems, the $\phi 16$ is equivalent to the diameter that 84 percent of the sample is finer (D84).

6.2.2 Native Beach Characterization. One sample of the native beach, near the east end of West Silver Sands Beach (South End Point), was collected from the intertidal zone and subjected to mechanical analysis. For this reconnaissance level effort, the gradation data for this sample are considered representative of the native beach. The grain size analysis data are shown in Figure B-12. The Corps beachfill specification envelopes are also plotted in this figure to allow comparison. Sample gradation data are shown below.

		ϕ Values		
D50 (mm) = 1.78 mm	= $\phi 50$	= -0.83		
D84 (mm) = 4.60 mm	= $\phi 16$	= -2.20	$M_{\phi n}$	= -0.6
D16 (mm) = 0.50 mm	= $\phi 84$	= 1.00	$S_{\phi n}$	= 1.6

6.2.3 Borrow Materials. Land-based sources were targeted primarily, but some general information about the potential for offshore sand source development is also provided. The quantity of sand required for the project ranges between 90,000 and 355,000 cubic yards, depending on the length of beach and the storm event for which protection is to be afforded.

6.2.3.1 Land-based Sources. A material survey was conducted of sand sources within a radius of approximately 80 miles of the study area, as sand has become increasingly more difficult to procure in southwestern Connecticut. Most of the sand produced in the immediate vicinity of the project is used for commercial cement/concrete production and for road sanding in the winter. Of the seven sources contacted within a 30-mile radius of the study area, only one (Beacon Falls) could potentially supply the quantity of sand required, by either trucking the material from eastern Connecticut, or by barging the material from New Jersey. Of the remaining six sources, two could not produce the required quantity (North Haven and Wallingford); two could not be reached or were otherwise uncooperative (North Haven and Hartford); one was cost prohibitive due to the specialized grading equipment used (Wallingford). The last source (Naugatuck) could probably produce the required quantity of sand, but the material may not be suitable for a beachfill application. This source crushes oversize materials from the bank deposit to make "concrete sand," which could result in an objectionable content of sharp rock fragments.

Five potential sources were identified at distances of greater than approximately 45 miles of the study area. The eastern Connecticut source mentioned above is located at a distance of approximately 50 miles (Jewett City), and is capable of producing the required quantity of sand. The second source, also located at a distance of approximately 50 miles in eastern Connecticut (Lisbon), would likely be capable of producing the quantity of sand required, but was still lacking one of the required permits for doing business. The third source, located at a distance of approximately 70 miles in northeastern Connecticut (Putnam) is also capable of producing the required quantity of sand. Two sources located in Massachusetts at distances of 80 miles or more are also capable of producing the required quantity of sand (Westfield and Falmouth).

At least two of the land-based sources mentioned (Jewett City, CT, and Falmouth, MA) have the capability to barge sand either directly to the study area or to New Haven, where it would be offloaded and trucked the remaining distance. Material barged from distant sources are economically competitive with material produced locally due to the high material costs in southwestern Connecticut. The distant sources which rely on trucking are also competitive due to their significantly lower material costs.

Typical grain size analysis results were obtained from the following sources: Naugatuck, Jewett City, Putnam, Westfield, MA, and Falmouth, MA. Gradations are plotted in Figures B-14 through B-18. The Corps beachfill specification envelope is also plotted in these figures for reference.

6.2.3.2 Offshore Sources. Offshore sources for suitable sand may exist. Generally, the continental shelf north of Chesapeake Bay, to the Maine - Canada border, is considered to have "promising" offshore sand mining potential (NOAA, 1977). While dredging costs are influenced by many factors (depth of water, shape and size of borrow area, grain size of material being dredged, etc.), dredging costs generally increase with distance from the shore. Dredging should generally be conducted at a distance far enough off shore, however, so that it does not result in back-erosion of the shore. Typically, analytical models must be used to determine this "safe" distance. The oceanfloor immediately offshore from the study area appears to be fairly rocky and resistant, given the number of rocks mapped within 2,000 feet to 3,000 feet of shore. While deposits of suitable sand are probably present farther offshore, explorations to find and characterize the extent and suitability of a specific borrow area are outside the scope of a reconnaissance level study. When economics make the mining of these deposits a viable option, industry will likely respond by exploiting this resource.

6.2.4 Suitability and Compatibility Evaluations. Corps guidance (EM 1110-2-3301) provides a method for evaluating the suitability and compatibility of potential borrow materials, as detailed below.

For this study, the gradations of the fine and coarse curves of the NED Corps specification are evaluated as potential borrow materials relative to the native material. The majority of the sources available in the area meet the "concrete sand" specification, which falls within the NED Corps specification envelope. Suitability of source materials will be based on a comparison with the specification envelopes.

The $M_{\phi n}$ and $S_{\phi n}$ values for the native beach sand and the $M_{\phi b}$ and $S_{\phi b}$ values for the potential borrow area (Corps specification) are used to calculate the following ratios:

$$\frac{S_{\phi b}}{S_{\phi n}} \quad \text{and} \quad \frac{M_{\phi b} - M_{\phi n}}{S_{\phi n}}$$

The values calculated above are plotted on special charts, which show the predicted overfill ratio (R_a) and the predicted renourishment factor (R_j).

6.2.4.1 Overfill Ratio (R_a). The ratios calculated above can be used to predict the overfill ratio, R_a, by consulting the chart in Figure B-18. If the proposed borrow material plots in the "stable" field, no overfill is indicated. The quantity of sand calculated should be the quantity provided to the site. If the material plots outside of the stable field, then an overfill ratio (R_a) will be indicated on the chart (USACE, 1984). For example, if 100,000 cubic yards of material was calculated as the required volume of fill, and an overfill ratio of 1.15 was indicated, then 115,000 cubic yards of sand should then <u>theoretically</u> be provided to the site to make up for the loss of less stable, incompatible materials. If the material has a predicted overfill ratio of greater than 10, it is considered unstable.

When the gradation data for the two extremes of the NED Corps specification envelope ("fine" and "coarse") are used to evaluate their compatibility as potential borrow materials, with respect to the native material, the following overfill ratios are predicted (see Figure B-18).

Borrow Material Gradation	Overfill Ratios, R_a
NED "Coarse Side"	1.25
NED "Fine Side"	Unstable

Based on this analysis, the closer the gradation of the borrow material is to the coarse side of the NED specification, the less material is predicted to be lost during the initial placement.

6.2.4.2 Renourishment Factor (R_j). The same values calculated (using $M_{\phi n}$, $M_{\phi b}$, $S_{\phi n}$, and $S_{\phi b}$) to predict the overfill ratio can be used to predict the renourishment factor (R_j) (Figure B-19) (USACE, 1984). The renourishment factor predicts how frequently renourishment will have to be provided for a borrow material, as compared to the native material. For example, an R_j value of 1.5 would indicate that renourishment would be required 1.5 times as often as the native material in order to maintain project dimensions. An R_j of 0.05 (1/20) would indicate that the selected borrow material would require renourishment one twentieth as often as the native material. This analysis does not calculate the frequency or quantity of required renourishment; it is a purely relative approach.

When this method is used to evaluate borrow performance using the two extremes of the Corps specification envelope ("fine" and "coarse") as the proposed borrow gradations, the following overfill ratios are predicted (see Figure B-19).

Borrow Material Gradation	Renourishment Factors, R_j

NED "Coarse Side"	1.2
NED "Fine Side"	4.0

Generally, the coarser the borrow material, the less frequently renourishment is predicted to be required.

Based on the analyses performed, sand falling within the coarser range of the Corps specification is predicted to perform more closely to the native material at West Silver Sands Beach. This makes sense, as the gradation of the native material practically falls along the curve for the coarse side of the Corps specification. Borrow materials coarser than the native material would tend to be even more stable, but are not commonly available. Most of the gradation data obtained from viable sources fell within the Corps envelope, although many tended towards the finer range of the envelope.

7. RECOMMENDED FUTURE STUDIES

Borings and/or test pits should be conducted as part of the Feasibility Study to facilitate the design of any dike or concrete wall warranting further study.

Additional sand samples should be collected from the existing beach and from potential sources, and tested (grain size analyses), for use in performing more detailed compatibility analyses, calculating required quantities, and in refining the material specification for any beachfill alternative warranting further study. If trucks are used for material delivery to the site, then a study of the local access roads should be conducted to verify that the existing roads can handle the additional truck traffic, or to determine how to mitigate the impacts of truck traffic.

REFERENCES

Bloom, A. L., 1967, Coastal Geomorphology of Connecticut, Geography Branch, Office of Naval Research, Contract No. 401(45), Task No. NR 388-065.

Brown, C. E., 1974a, Map Showing Unconsolidated Materials, Branford Quadrangle, Connecticut, United States Geological Survey Miscellaneous Field Studies Map MF-560A.

Brown, C. E., 1974b, Contour Map of the Bedrock Surface, Branford Quadrangle, Connecticut, United States Geological Survey Miscellaneous Field Studies Map MF-560C.

Flint, R. F., 1964, The Surficial Geology of the Branford Quadrangle, Connecticut (with Map), State Geological and Natural History Survey of Connecticut, Quadrangle Report No. 14.

Krumbein, W. C., and Sloss, L. L., 1963, Stratigraphy and Sedimentation, W. H. Freeman and Company, San Francisco, CA.

Mikami, H. M., and Digman, R. E., 1957, The Bedrock Geology of the Guilford 15-Minute Quadrangle and a Portion of the New Haven Quadrangle, State Geological and Natural History Survey of Connecticut, Bulletin No. 86.

Outer Continental Shelf Policy Committee, 1993, "U.S. Outer Continental Shelf Sand and Gravel Resources," Final Report of the OCS Policy Committee's Subcommittee on OCS Sand and Gravel Resources, April, 1993.

Padan, J. W., ed., 1977, New England Offshore Environmental Study, U.S. Department of Commerce, National Oceanic and Atmospheric Administration, Environmental Research Laboratories, Special Report.

Rodgers, J., 1985, Bedrock Geological Map of Connecticut, Connecticut Geological and Natural History Survey (1:125,000).

U.S. Army Corps of Engineers, 1984, Shore Protection Manual, Coastal Engineering Research Center, Waterway Experimentation Station, Vicksburg, MS.

U.S. Army Corps of Engineers, 1995, "Design of Beach Fills," Engineer Manual 1110-2-3301.

U.S. Department of Commerce, National Oceanic and Atmospheric Administration, 1988, Sea Level Variations for the United States 1855-1986, National Ocean Service, Rockville, MD.

FIGURES

FIGURE B-1

FIGURE B-2

FIGURE B-3

FIGURE B-4

FIGURE B-10
CORPS BEACHFILL SPECIFICATION

FIGURE B-11
CT DOT "CONCRETE SAND" SPECIFICATION

FIGURE B-12
NATIVE BEACH SAMPLE GRADATION
WEST SILVER SANDS BEACH, CT

FIGURE B-13
NAUGATUCK SOURCE GRADATION

FIGURE B-14
JEWETT CITY SOURCE GRADATION

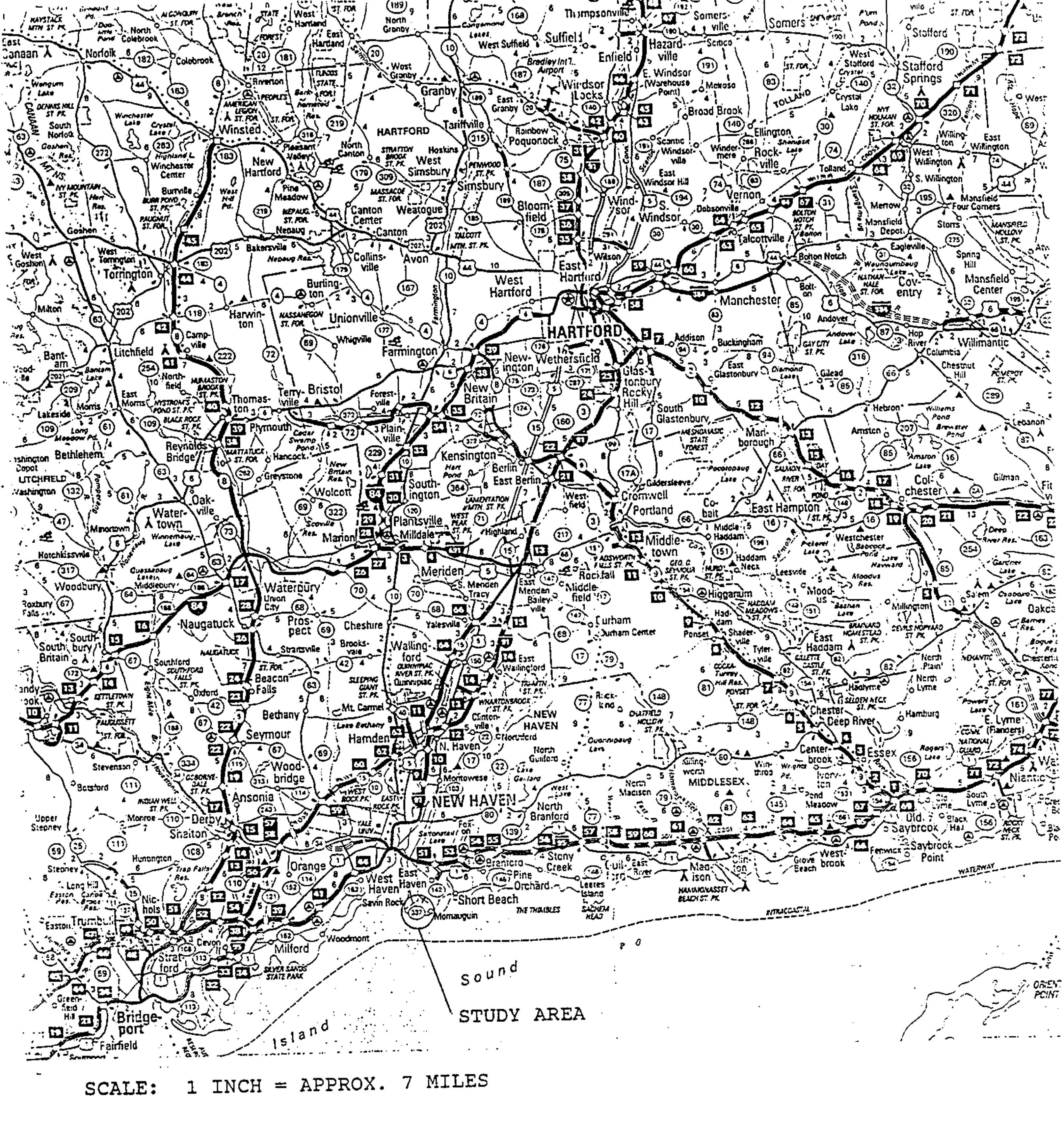
FIGURE B-15
PUTNAM SOURCE GRADATION

FIGURE B-16
WESTFIELD, MA SOURCE GRADATION

FIGURE B-17
FALMOUTH, MA SOURCE GRADATION

FIGURE B-18
OVERFILL RATIO (R_a) CHART

FIGURE B-19
RENOURISHMENT FACTOR (R_j) CHART



SCALE: 1 INCH = APPROX. 7 MILES

FIGURE B-1
SECTION 103
WEST SILVER SANDS BEACH
EAST HAVEN, CONNECTICUT

SITE LOCATION MAP

MESOZOIC	JURASSIC TRIASSIC	YOUNGEST
PALEOZOIC	PERMIAN DEVONIAN SILURIAN ORDOVICIAN	
PROTEROZOIC		OLDEST

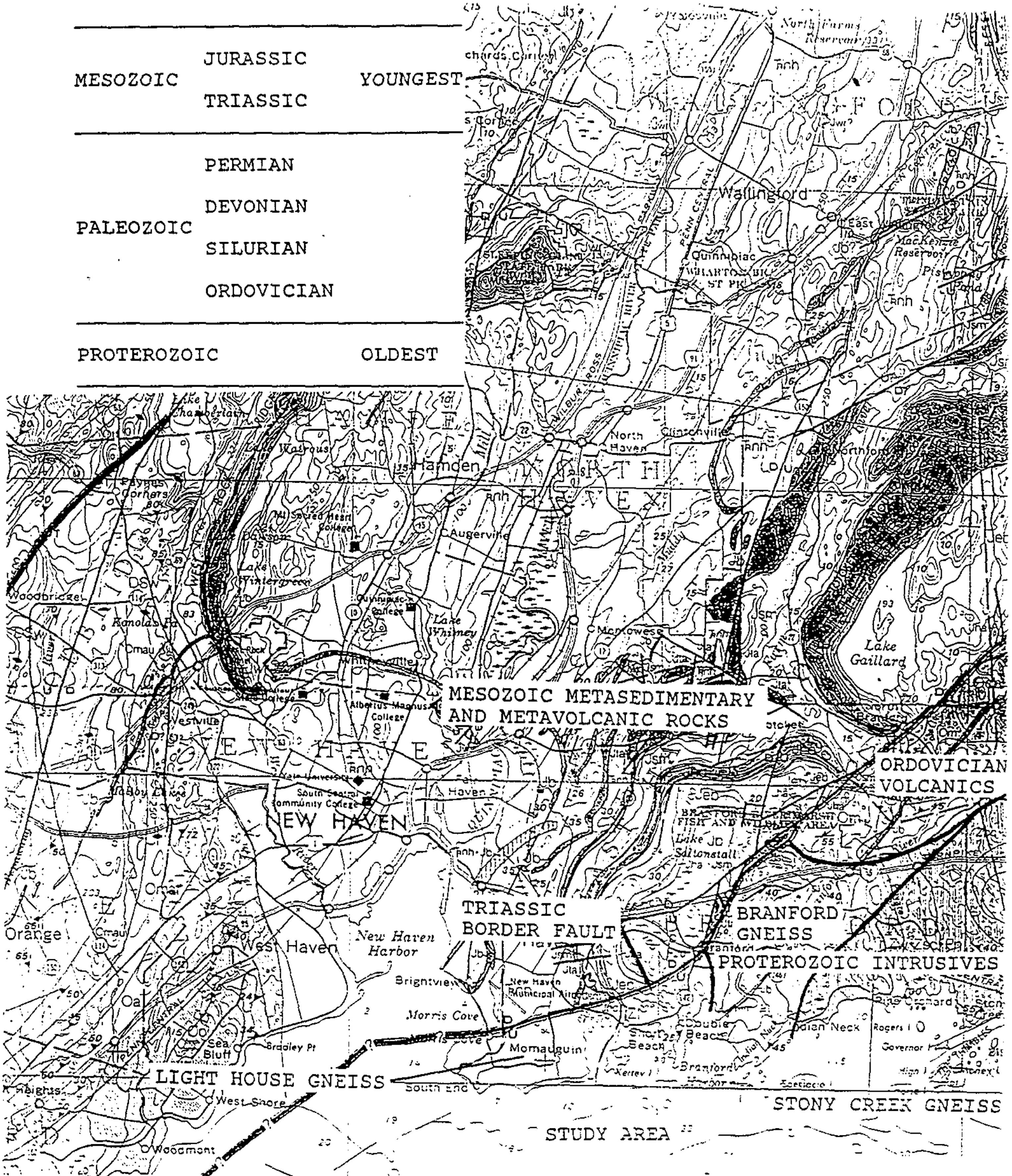


FIGURE B-2
SECTION 103
WEST SILVER SANDS BEACH
EAST HAVEN, CONNECTICUT

SCALE: 1 INCH = APPROX. 2 MILES

BEDROCK GEOLOGY MAP

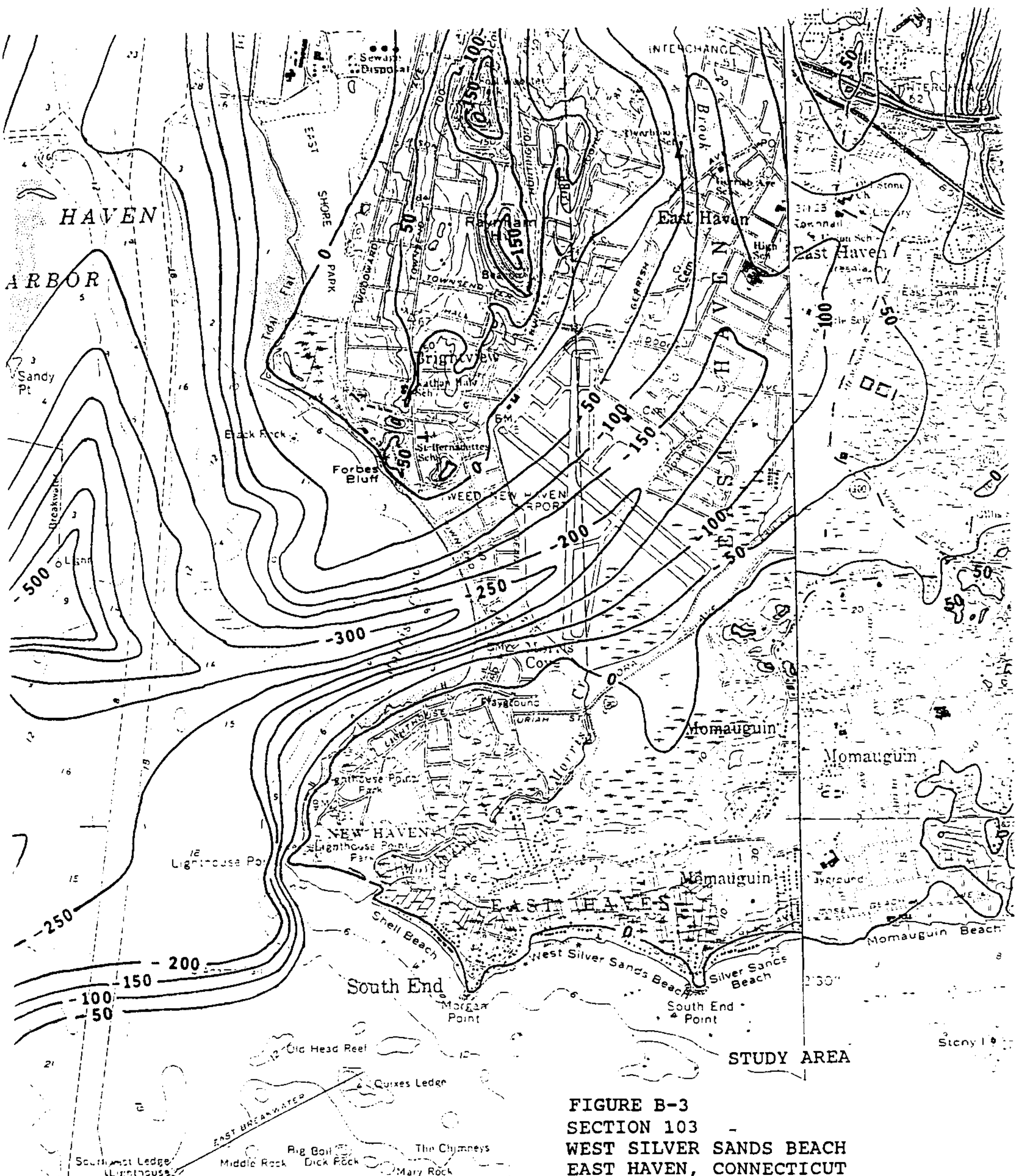


FIGURE B-3
SECTION 103 -
WEST SILVER SANDS BEACH
EAST HAVEN, CONNECTICUT

BEDROCK CONTOUR MAP

DATUM = MEAN SEA LEVEL
CONTOUR INTERVAL = 50 FEET
SCALE: 1 : 24,000
(1 INCH = APPROX. 2,000 FEET)

STUDY AREA

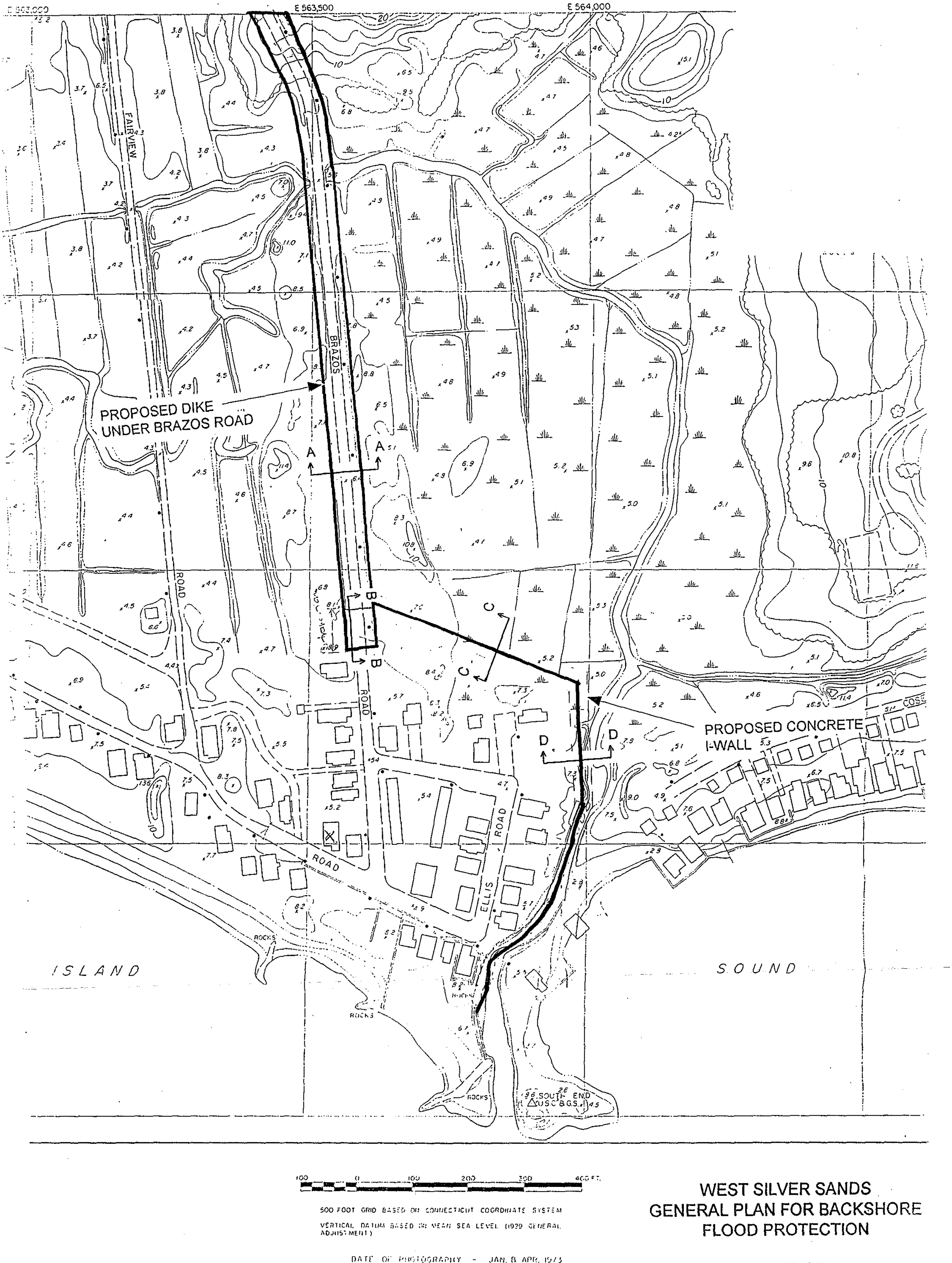
FIGURE B-4

SECTION 103
WEST SILVER SANDS BEACH
EAST HAVEN, CONNECTICUT

SURFICIAL GEOLOGY MAP

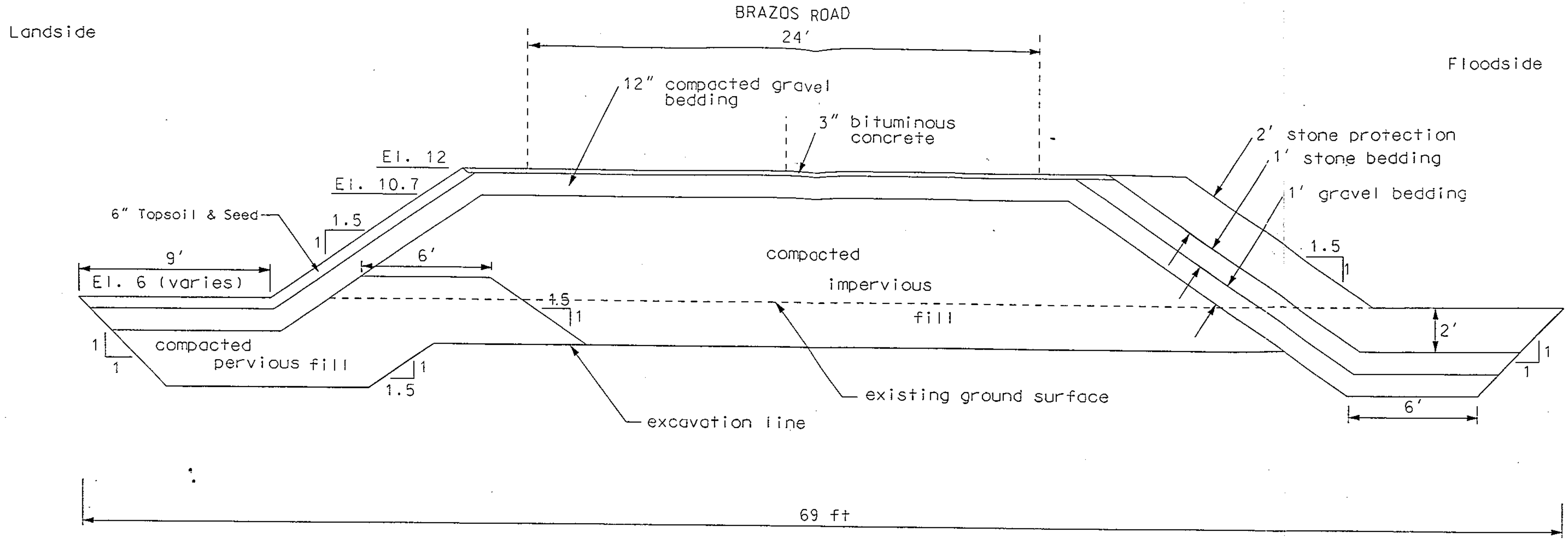
SCALE: 1 : 24,000
(1 INCH = APPROX. 2,000 FEET)

BEDROCK
OUTCROPS



WEST SILVER SANDS
GENERAL PLAN FOR BACKSHORE
FLOOD PROTECTION

FIGURE B-5



SECTION A-A

CROSS SECTION OF RAISED BRAZOS ROAD
100 YEAR STORM PROTECTION
scale : 1" = 5'

NOTE:

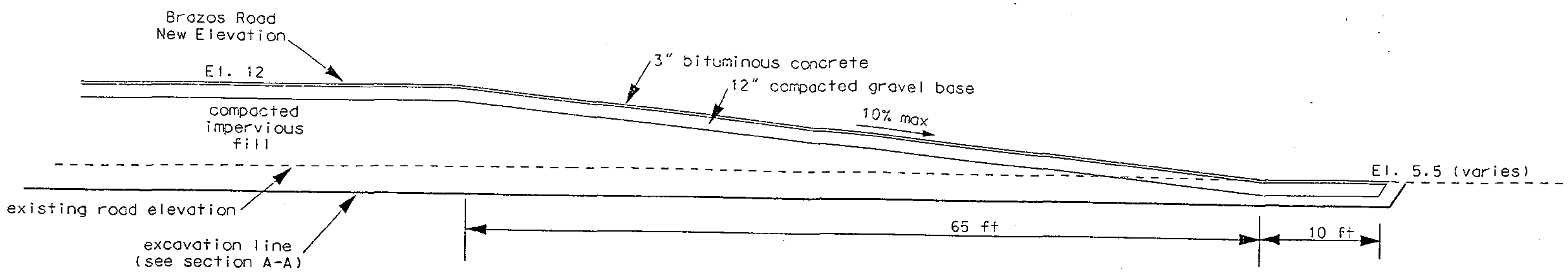
All Elevations are in feet-NGVD

TYPICAL DIKE SECTION
SECTION A-A

SECTION 103
WEST SILVER SANDS BEACH

filename: DIKE2.dgn | E. HAVEN, CT

NOVEMBER 1995 | FIGURE B-6



PROFILE OF BRAZOS ROAD
Transition from existing elevation
to raised elevation

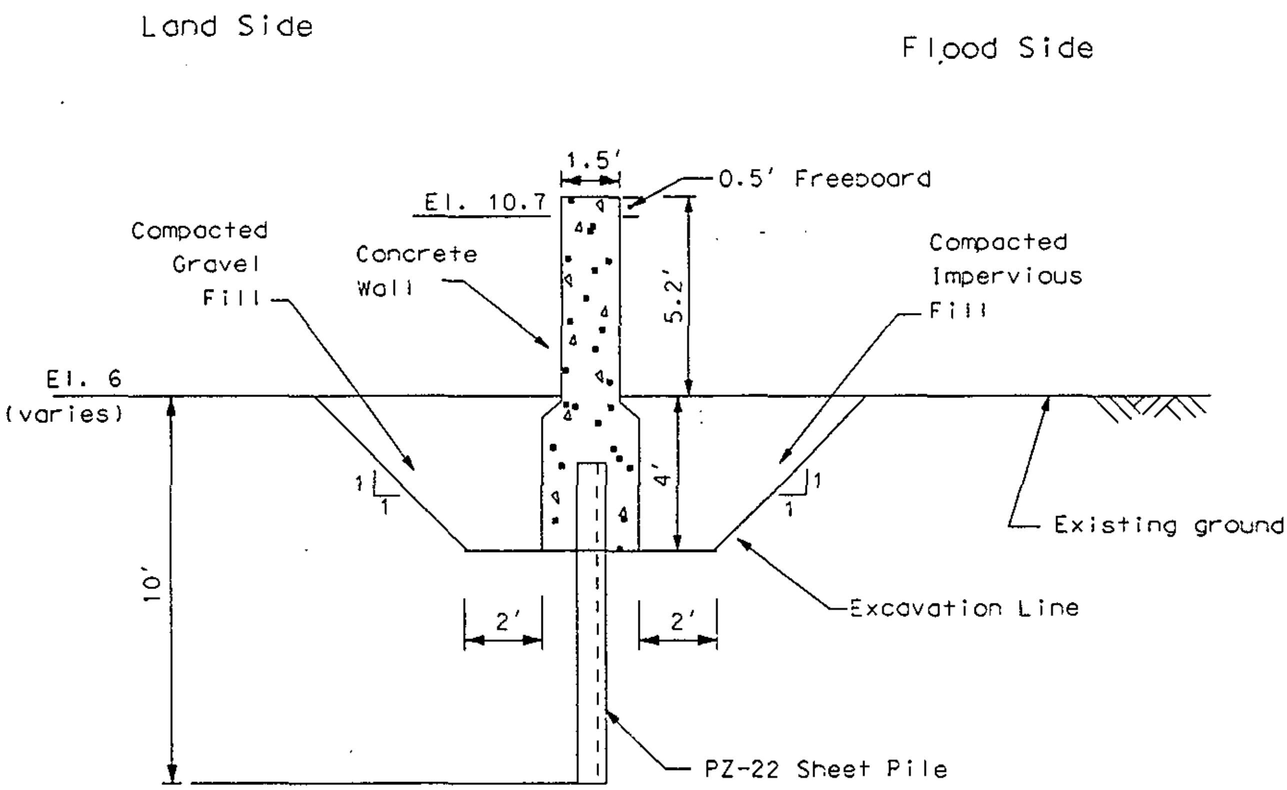
Scale: 1" = 10'

SECTION B-B

NOTE:

All Elevations are in feet-NGVD

NEW ROAD PROFILE AND TRANSITION SECTION B-B SECTION 103 WEST SILVER SANDS BEACH	
filename: DIKE2.dgn	E. HAVEN, CT
NOVEMBER 1995	FIGURE B-7



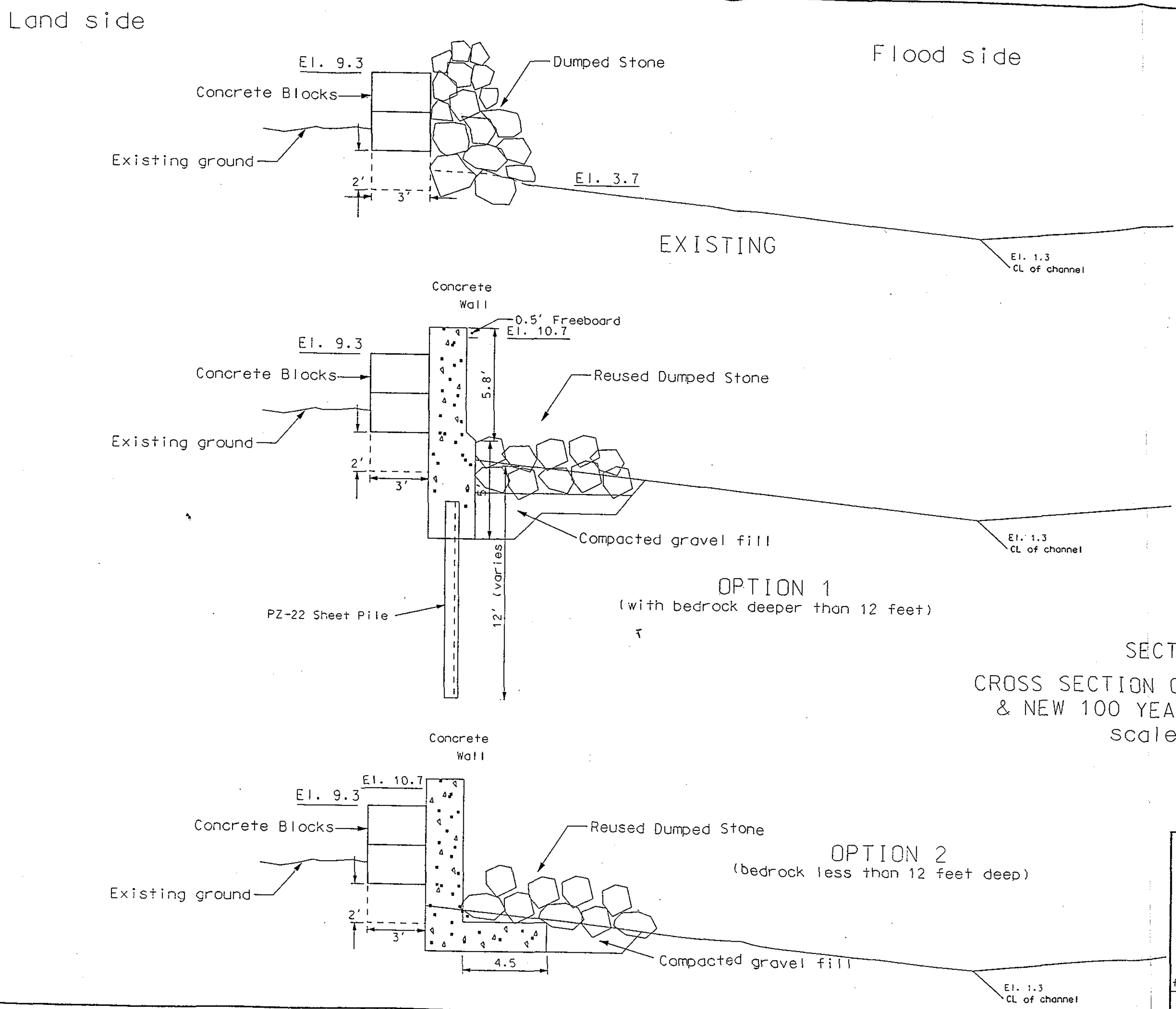
CROSS SECTION OF FLOOD WALL
100 YEAR STORM PROTECTION
Scale: 1" = 5'
SECTION C-C

NOTE:
All elevations are ft-NGVD

TYPICAL MARSH WALL SECTION
SECTION C-C

SECTION 103
WEST SILVER SANDS BEACH

filename: DIKE2.dgn E. HAVEN, CT
NOVEMBER 1995 FIGURE B-8



SECTION D-D

CROSS SECTION OF EXISTING CONCRETE
& NEW 100 YEAR STORM PROTECTION
scale : 1" = 5'

NOTE:

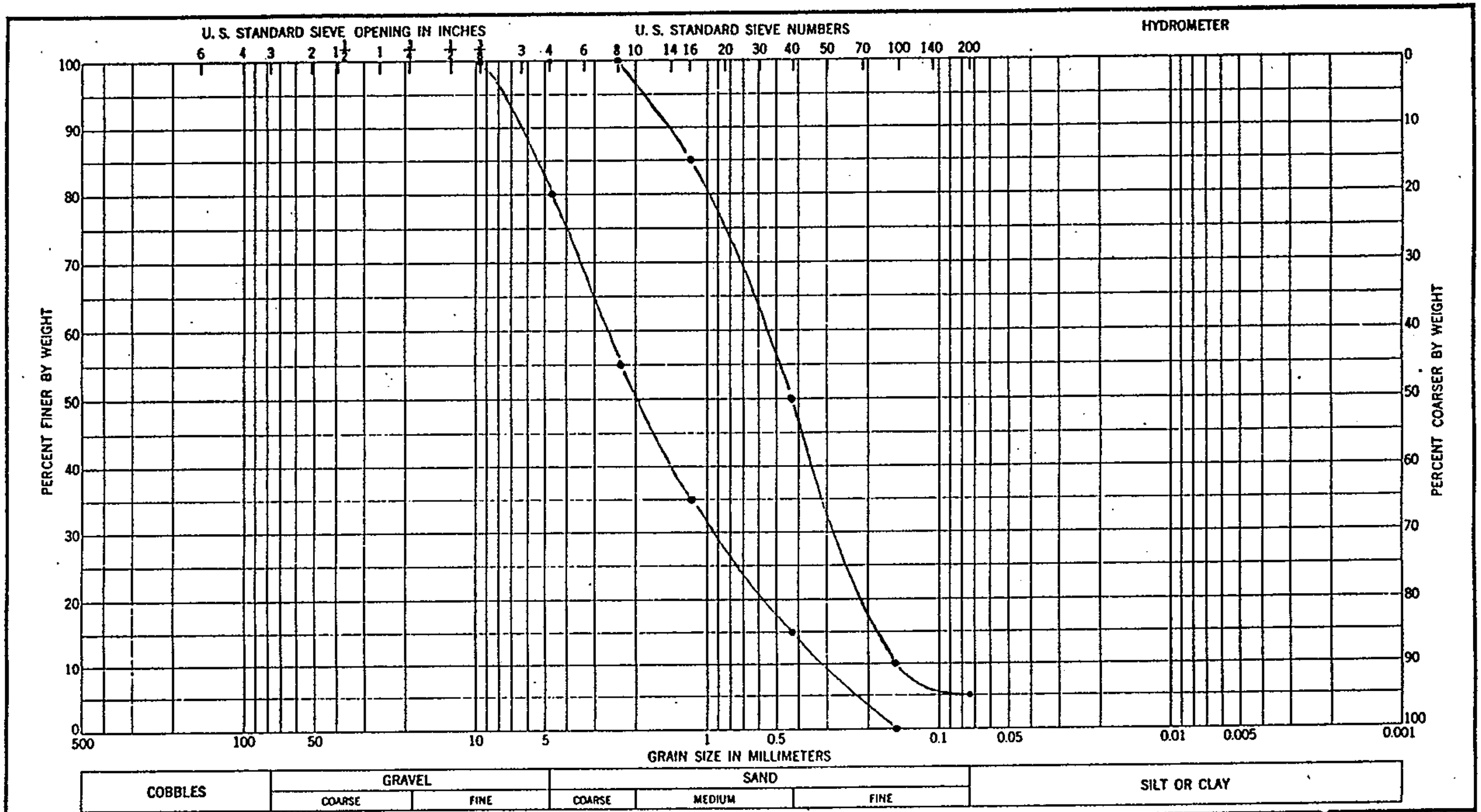
All Elevations are in feet-NGVD

TYPICAL SEASIDE
WALL SECTION D-D

SECTION 103
WEST SILVER SANDS BEACH

filename: DIKE2.dgn | E. HAVEN, CT

NOVEMBER 1995 | FIGURE B-9



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
	—•—	*Beachfill Specification*				

Project

Area

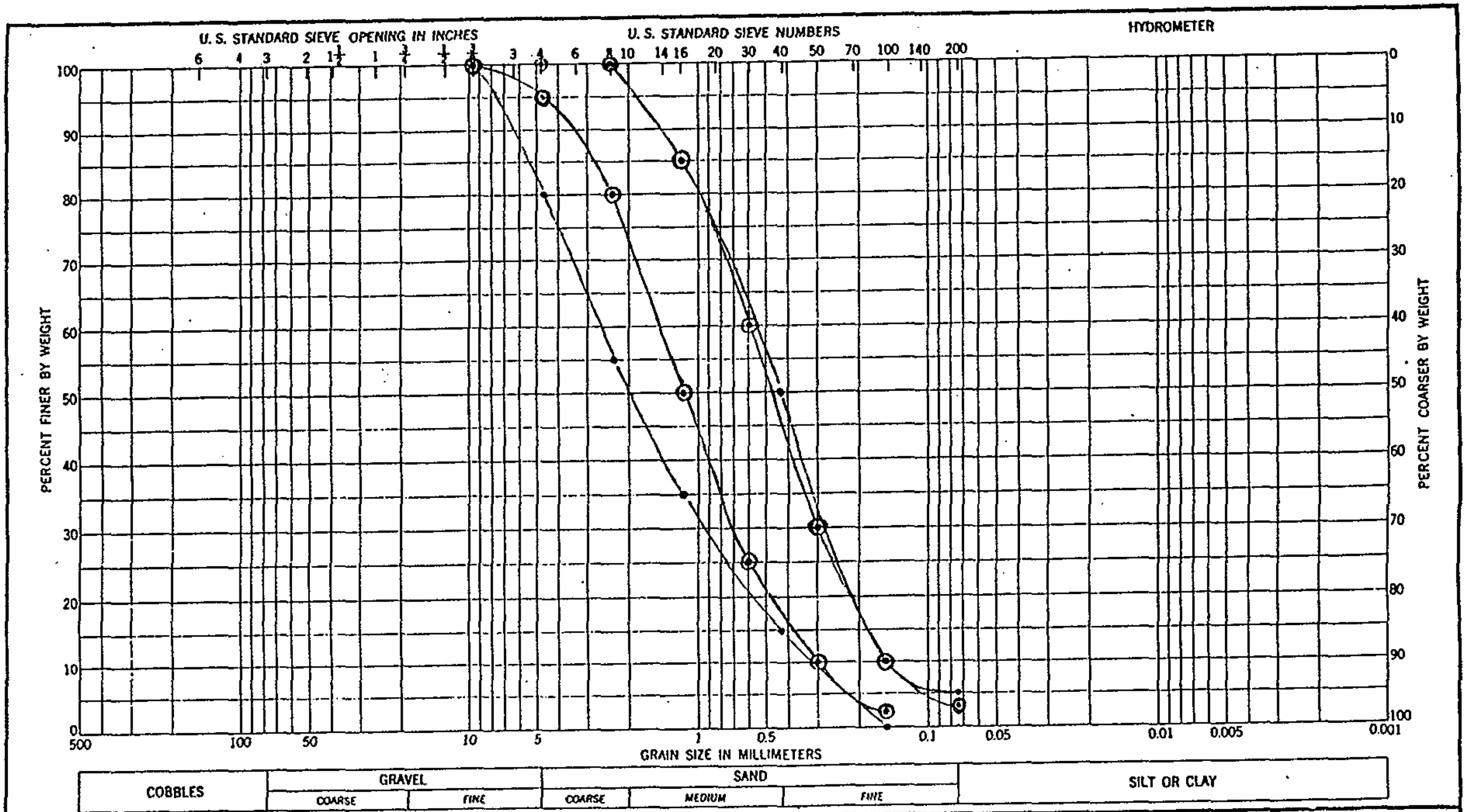
Boring No.

Date

GRADATION CURVES

ENG FORM 1 MAY 63 2087

FIGURE B-10
CORPS BEACHFILL SPECIFICATION



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
	-•-	Beachfill Specification				
	-⊙-	CT DOT "Concrete Sand"				

Project

Area

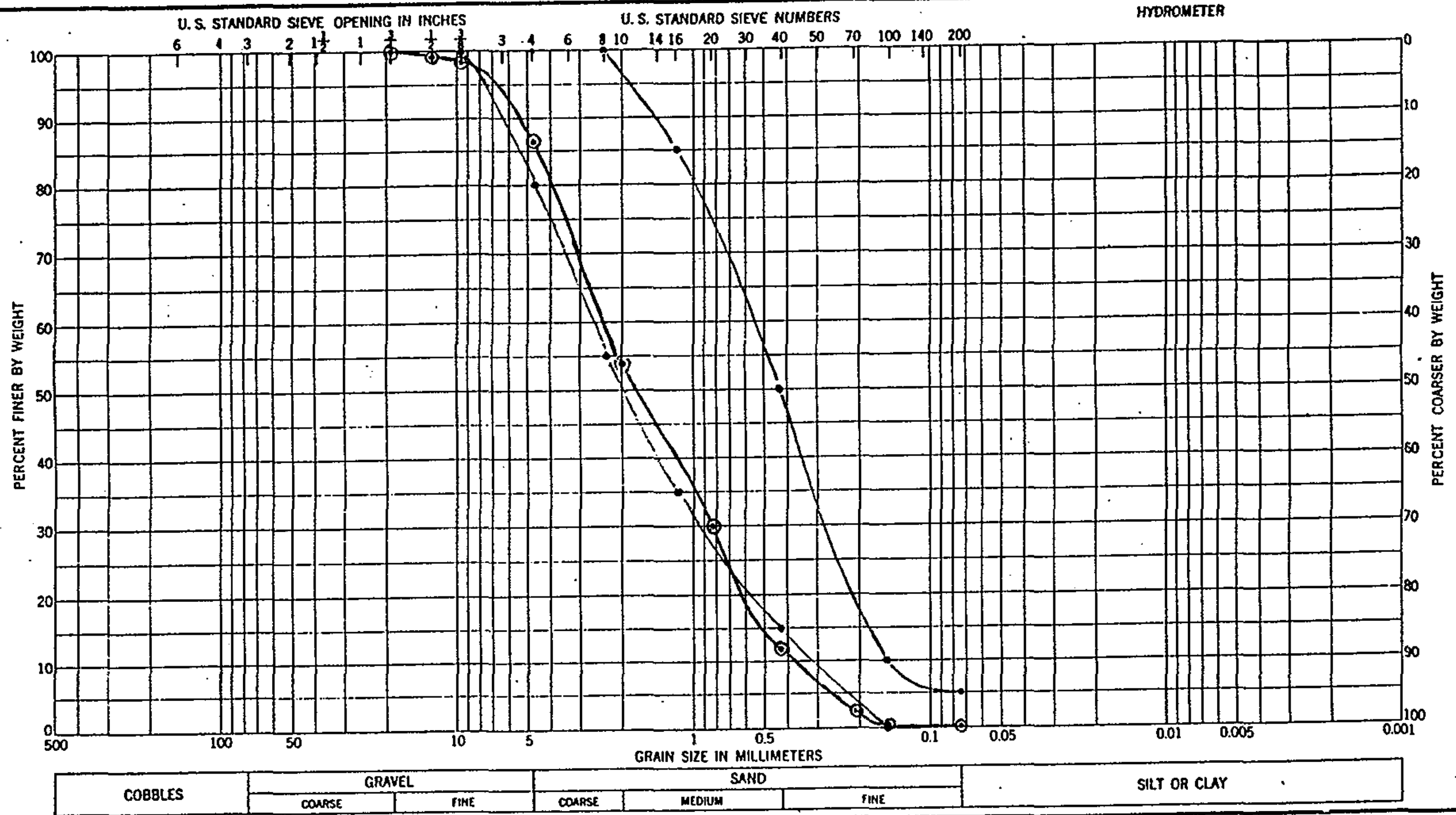
Boring No.

Date

GRADATION CURVES

ENG FORM 1 MAY 63 2087

FIGURE B-11
CT DOT "CONCRETE SAND" SPECIFICATION



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
	—•—	*Beachfill Specification*				
	—⊙—	*Native Beach Sample*				
		W. Silver Sands Beach				

Project

Area

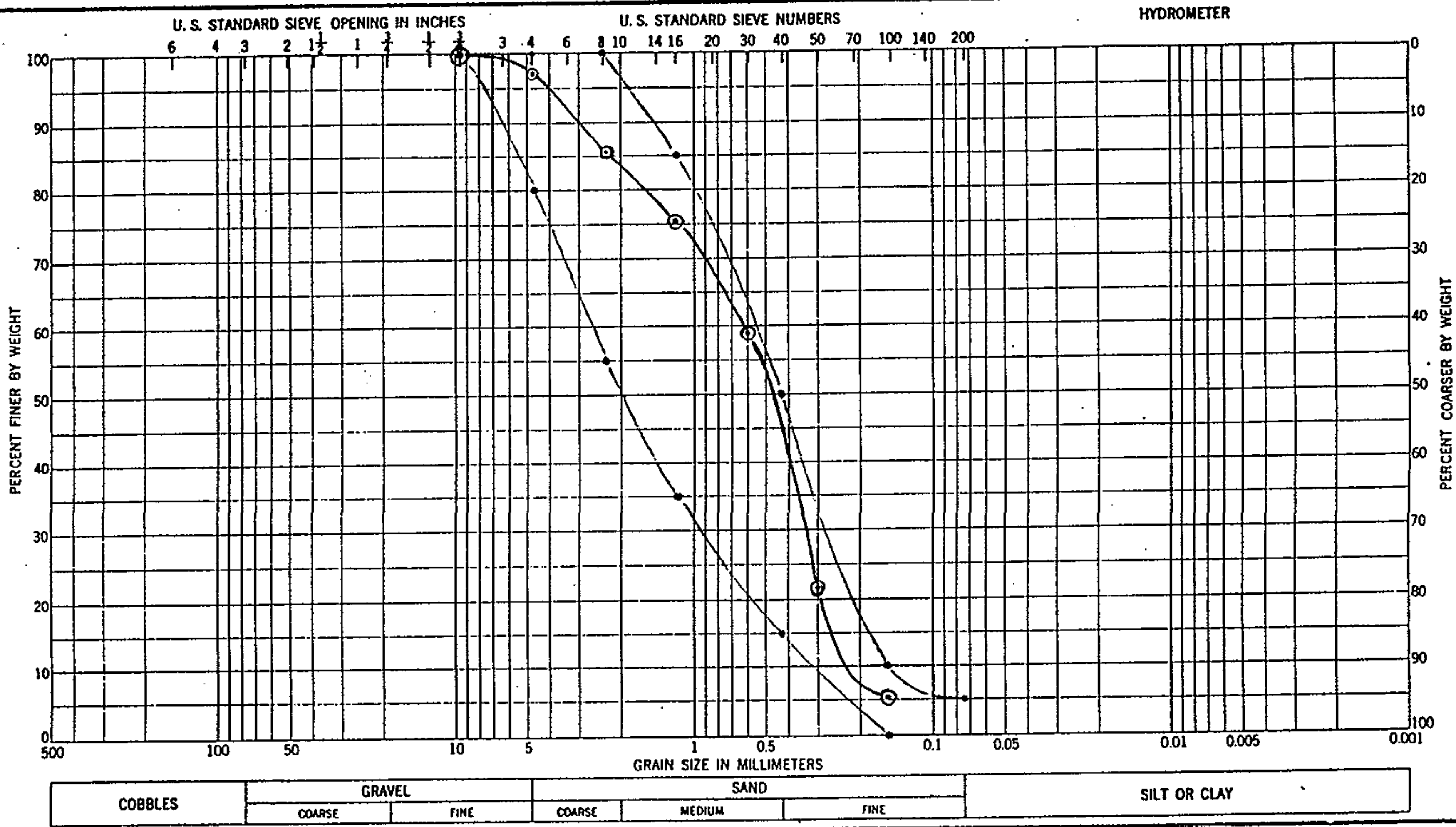
Boring No.

Date

GRADATION CURVES

ENG FORM 1 MAY 63 2087

FIGURE B-12
NATIVE BEACH SAMPLE GRADATION
WEST SILVER SANDS BEACH, CT



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
	—•—	Beachfill Specification				
	—⊙—	Naugatuck, CT				
		Test Data Received 11-17-95				

Project

Area

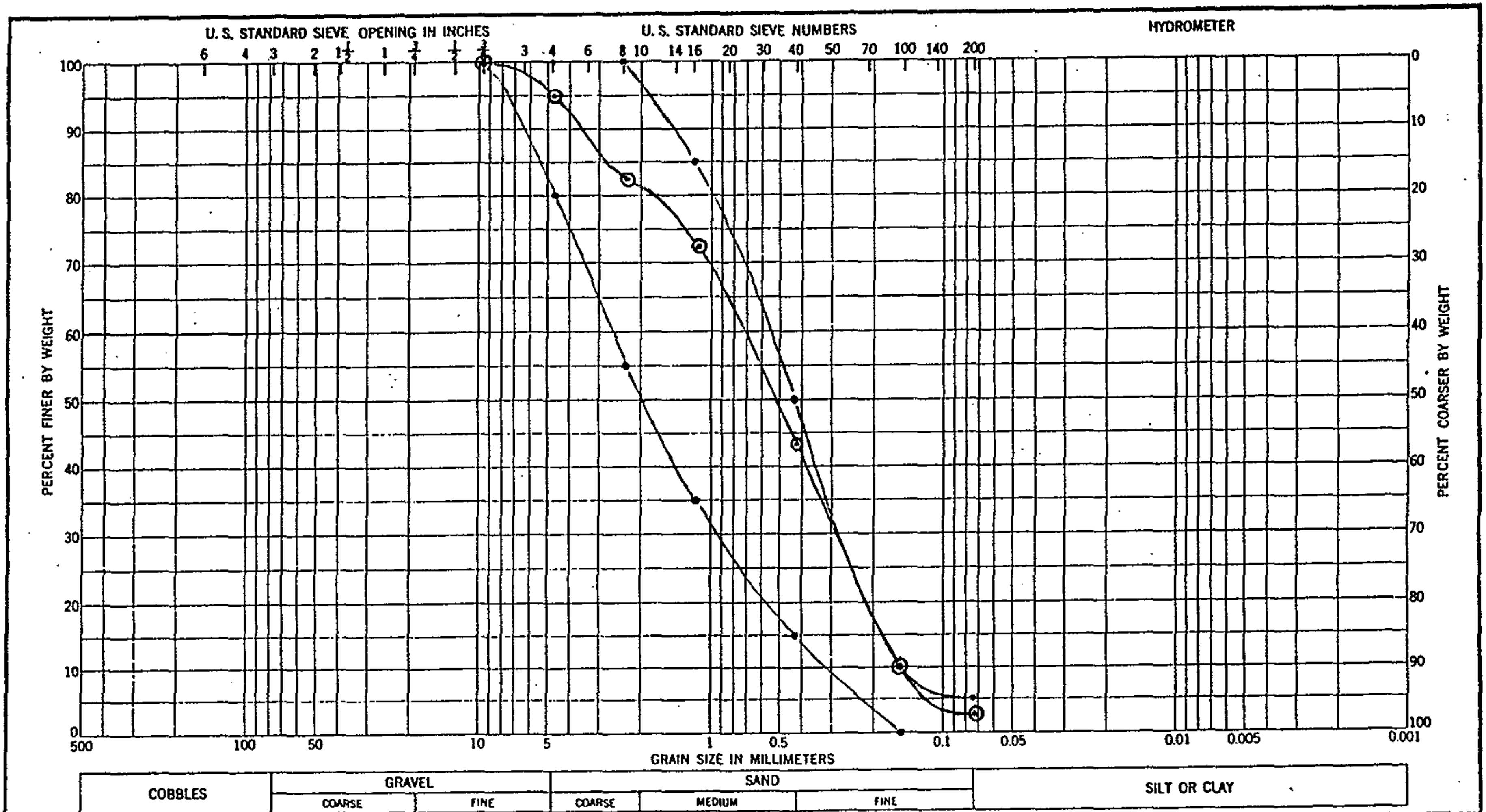
Boring No.

Date

GRADATION CURVES

ENG FORM 1 MAY 63 2087

FIGURE B-13
NAUGATUCK SOURCE GRADATION



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
	-•-	Beachfill Specification				
	-⊙-	Jewett City, CT				
		Test data submitted 5/6/95				

Project

Area

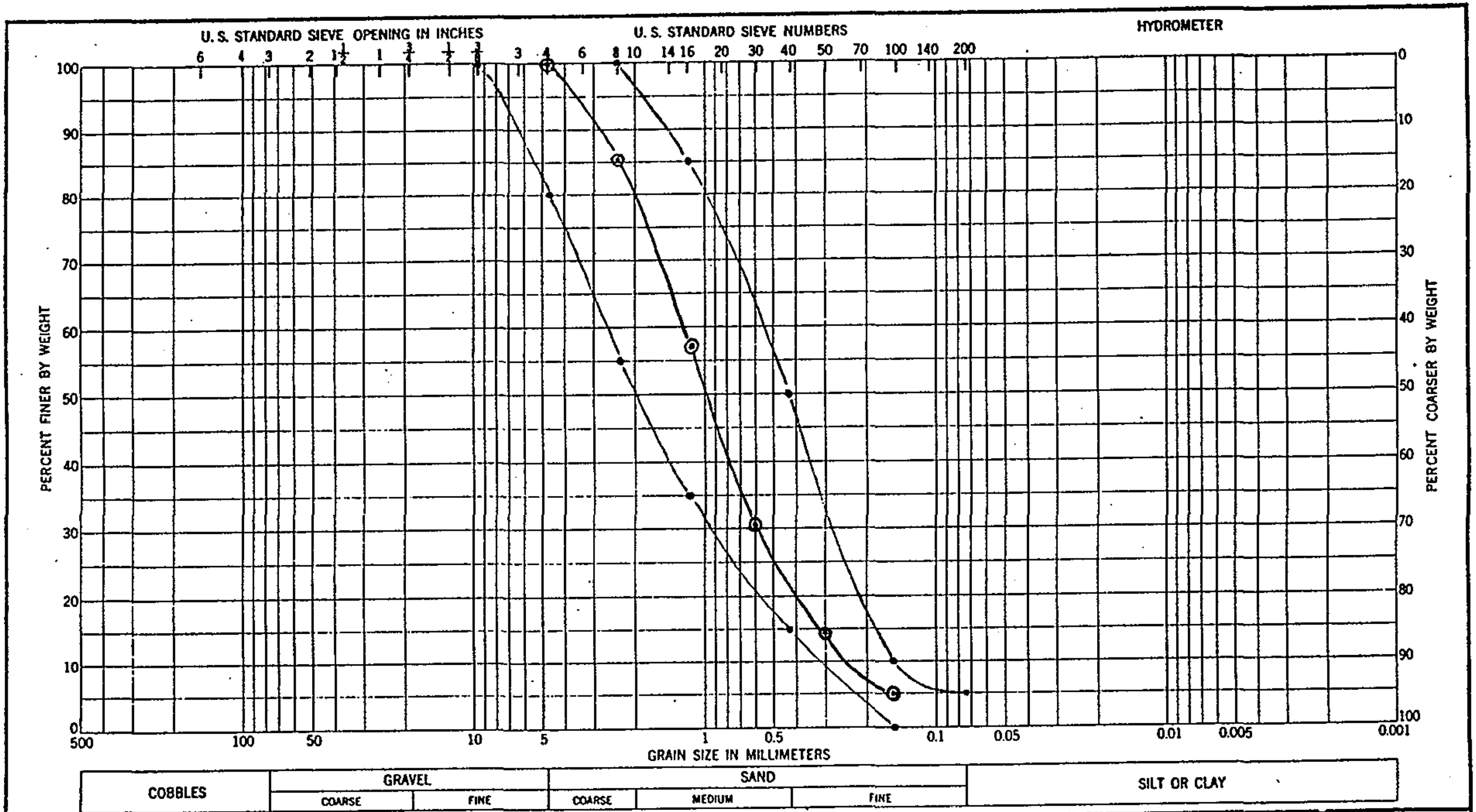
Boring No.

Date

GRADATION CURVES

ENG FORM 1 MAY 63 2087

FIGURE B-14
JEWETT CITY SOURCE GRADATION



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
	—•—	Beachfill Specification				
	—⊙—	Putnam, CT				

Project

Area

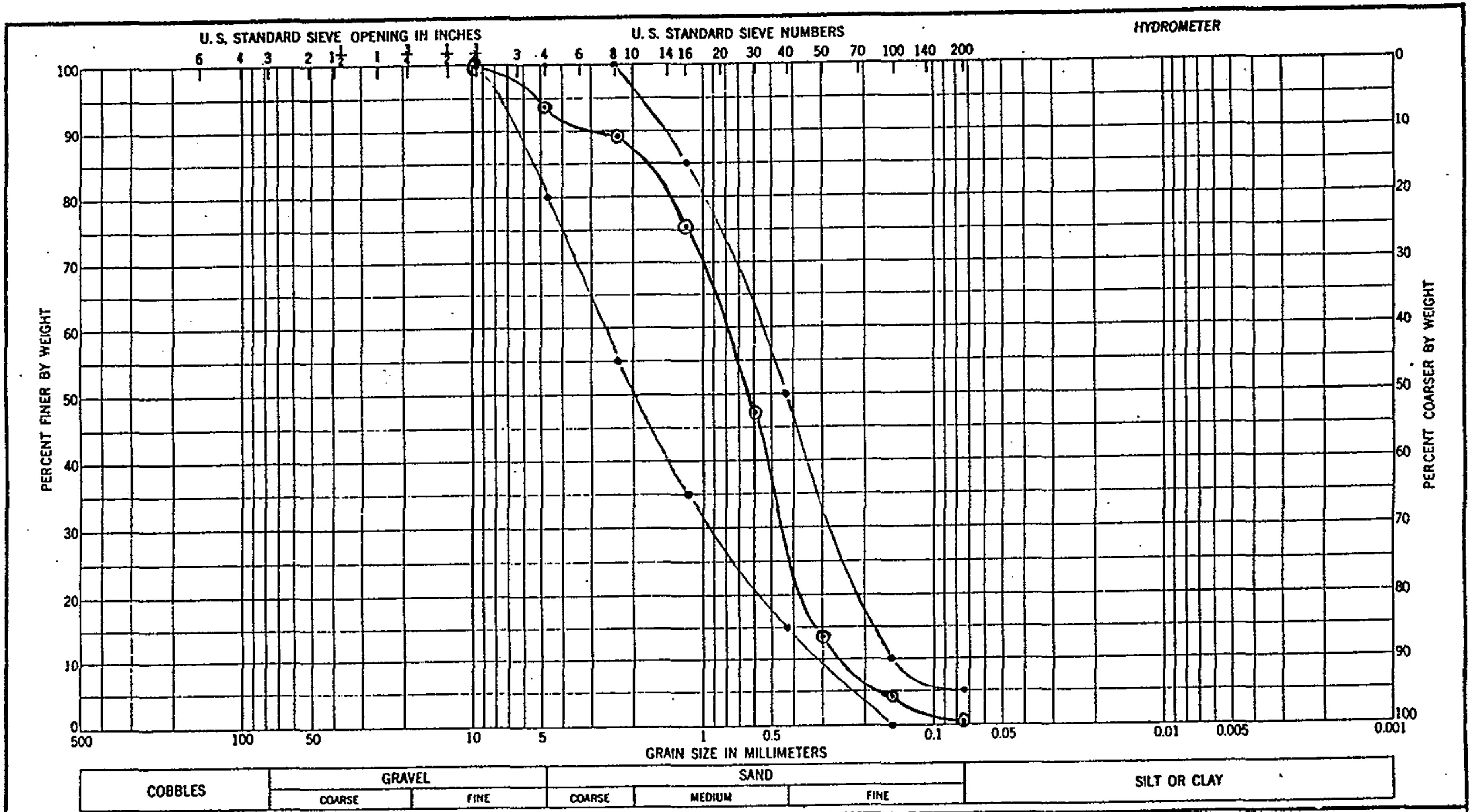
Boring No.

Date

GRADATION CURVES

ENG FORM 1 MAY 63 2087

FIGURE B-15
PUTNAM SOURCE GRADATION



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
	-•-	Beachfill Specification				
	-⊙-	Westfield, MA				
		Test Data 7-18-94				

Project

Area

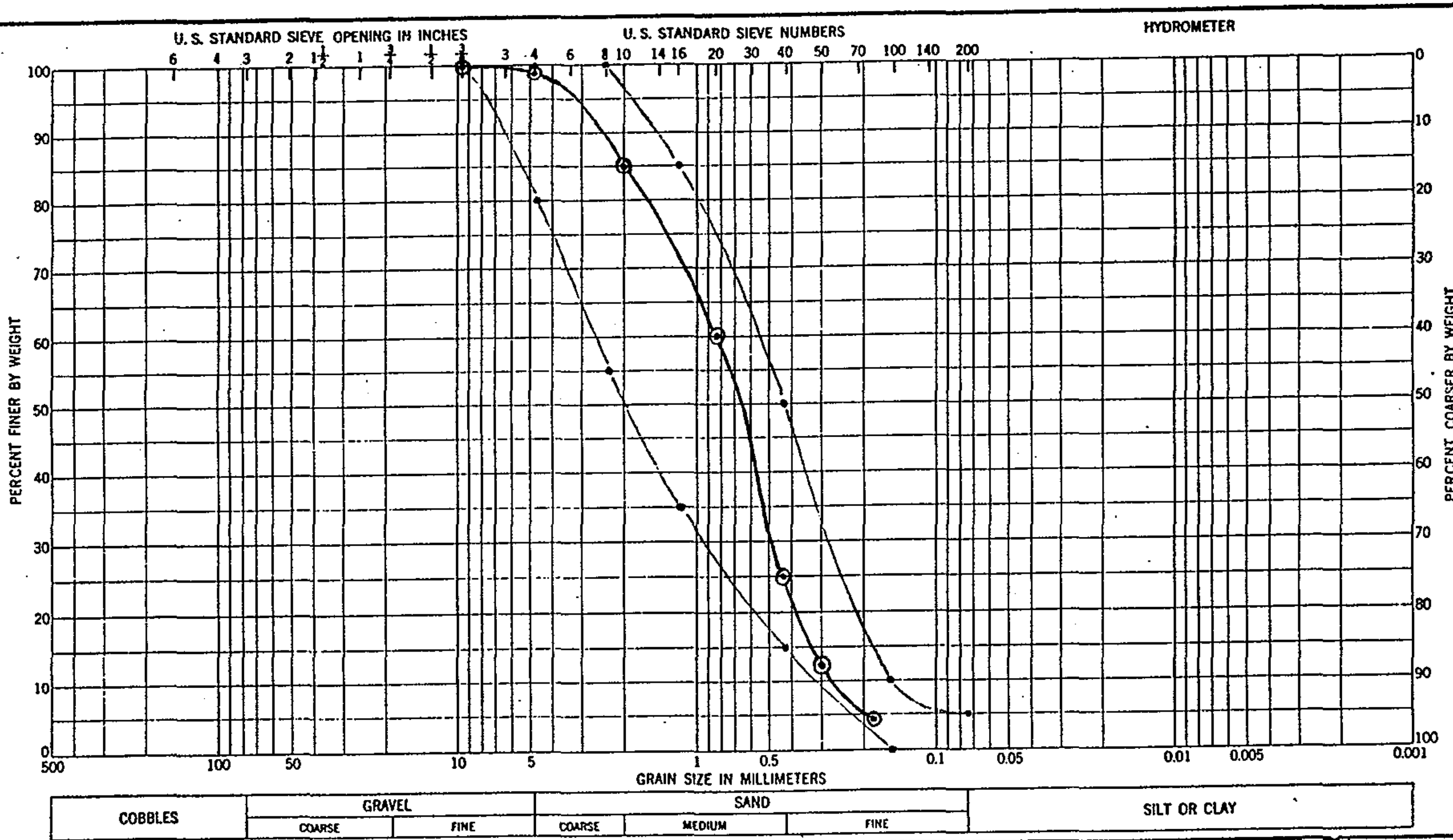
Boring No.

Date

GRADATION CURVES

ENG FORM 1 MAY 63 2087

FIGURE B-16
WESTFIELD, MA SOURCE GRADATION



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
		Beachfill Specification				
		Falmouth, MA				

Project

Area

Boring No.

Date

GRADATION CURVES

ENG FORM 1 MAY 63 2087

FIGURE B-17
FALMOUTH, MA SOURCE GRADATION

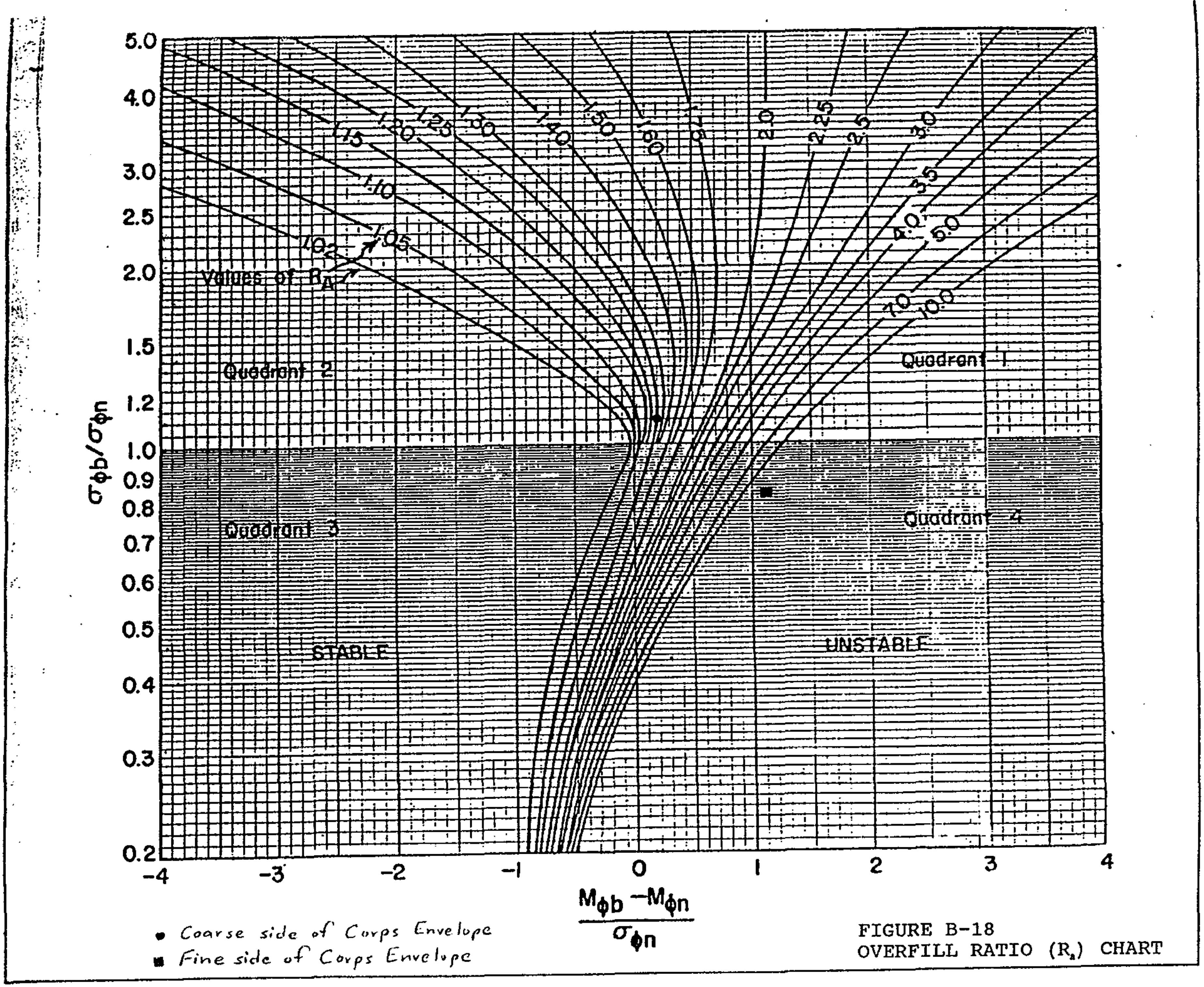
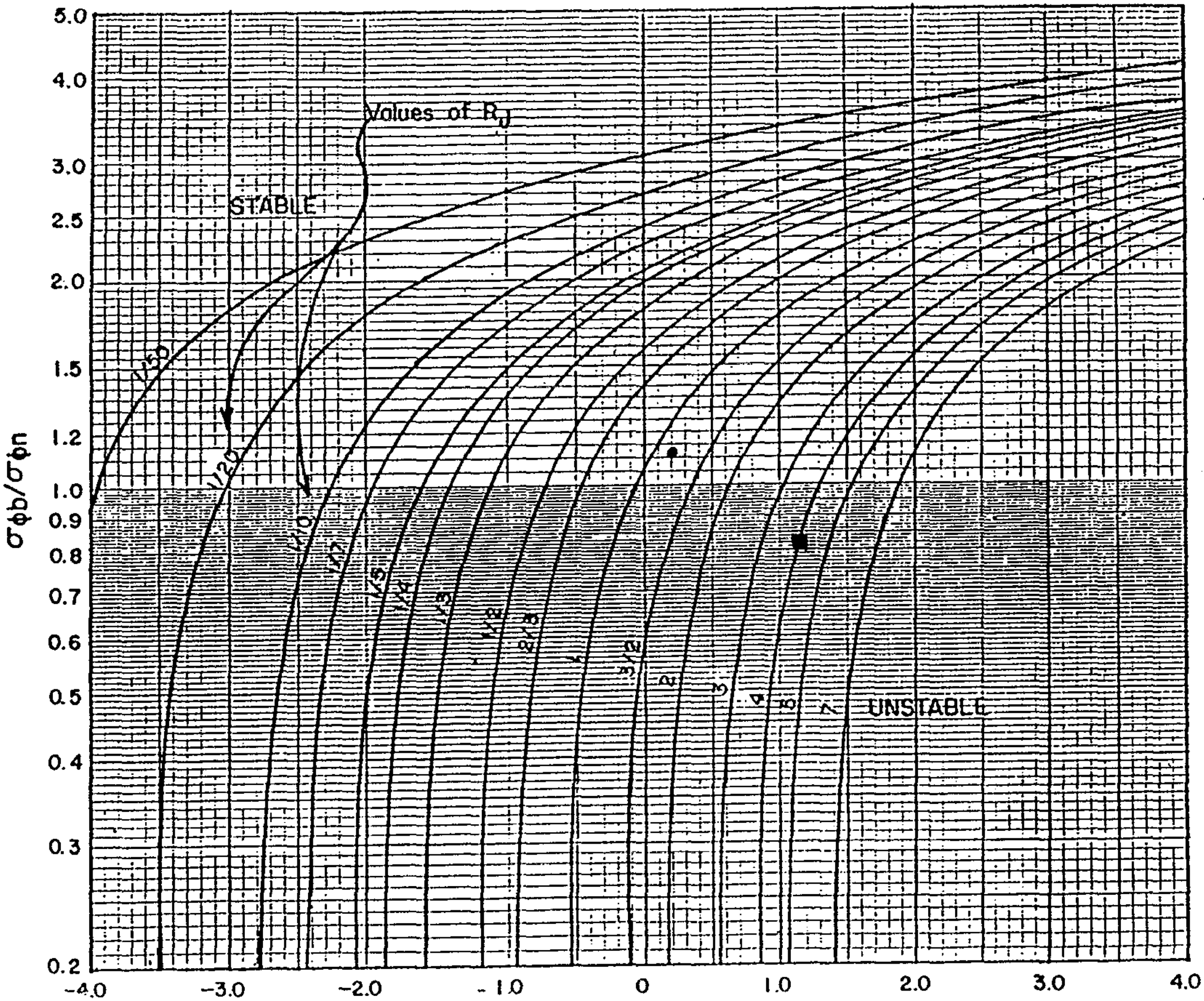


Figure 3-7. Isolines of the adjusted overfill ratio (R_A) for values of ϕ mean differences and ϕ sorting ratio (*Shore Protection Manual* 1984)



• Coarse side of Corps Envelope

■ Fine side of Corps Envelope

$$\frac{M_{\phi b} - M_{\phi n}}{\sigma_{\phi n}}$$

FIGURE B-19
RENOURISHMENT FACTOR (R_j) CHART

APPENDIX C
HYDROLOGY AND HYDRAULICS

APPENDIX C

HYDROLOGY AND HYDRAULICS

COASTAL FLOOD REDUCTION STUDY
WEST SILVER SANDS BEACH
EAST HAVEN, CONNECTICUT

PREPARED BY
ENVIRONMENTAL ENGINEERING AND
HYDRAULICS BRANCH
WATER CONTROL DIVISION
ENGINEERING DIRECTORATE

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MA 02254-9149

JANUARY 1996

APPENDIX C

HYDROLOGY AND HYDRAULICS

COASTAL FLOOD REDUCTION STUDY
WEST SILVER SANDS BEACH
EAST HAVEN, CONNECTICUT

TABLE OF CONTENTS

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF PLATES

APPENDIX C

HYDROLOGY AND HYDRAULICS

COASTAL FLOOD REDUCTION STUDY
WEST SILVER SANDS BEACH
EAST HAVEN, CONNECTICUT

1. SUMMARY

This report presents the results of studies concerning coastal flooding conditions of the low lying neighborhood behind West Silver Sands Beach, in East Haven, Connecticut. The area is bounded by an eroding beach to the south and west, a tidal creek (with revetment along the west bank) to the east, and a tidal marsh to the north. According to town officials and area residents, flooding in this area is caused mainly by tides in the creek, which enter the neighborhood at termination of the revetment. The purpose of the study was to evaluate flooding from the creek and from overtopping of the beach. The study, which includes climatology, rainfall analysis, and tidal investigations, concludes that flooding occurs from both the tidal creek and overtopping of West Silver Sands Beach. In order to prevent flooding of houses behind the beach, protection is needed on the beach as well as in the back- shore area.

2. DESCRIPTION

East Haven is an urban area, located in southern Connecticut, on Long Island Sound (see plates C-1 and C-2). The study area focuses on West Silver Sands Beach, located in the southern portion of East Haven (see plate C-3). The neighborhood behind West Silver Sands Beach consists of 40 to 50 low lying homes, which have received first floor flooding in the past. Brazos and Fairview Roads, leading to the neighborhood, have approximate elevations of 5.6 and 4.6 feet NGVD, respectively. According to area residents, these roads are impassible 20 times per year. West Silver Sands Beach has an average crest elevation of 8.4 feet NGVD in the study area. A wall that lines the tidal creek for 150 feet, has an elevation of 9 feet NGVD. This wall is fronted by rock revetment which has an approximate elevation of 11 feet NGVD. The marsh behind the houses ranges in elevation from 3 to 5 feet NGVD.

3. CLIMATOLOGY

a. General. The study area has a temperate and changeable climate characterized by four distinct seasons.

Due to the moderating influences of Long Island Sound and the Atlantic Ocean, and particularly changing movements of high and low pressure systems approaching from the west and southwest, extremes of either hot or cold weather are rarely of long duration. In winter, coastal storms frequently bring rainfall to the shore areas. The prevailing winds are northerly in the fall and winter, northwesterly in spring, and southerly in summer. High winds, heavy rainfall, and abnormally high tides occur with unpredictable frequency. Hurricanes occur most frequently during August, September, and October.

b. Temperature. Since 1896 temperature records have been maintained at Bridgeport, about 20 miles west of East Haven. The mean annual temperature is approximately 51 degrees Fahrenheit, with January and February the coldest months averaging about 30 degrees, and July the warmest at about 73 degrees. Freezing temperatures are common from late November through March. Table C-1 is a summary of the mean monthly, maximum, and minimum temperatures recorded at the Bridgeport National Weather Service station. Temperatures are based on the 98-year period of record from 1896 to 1993.

c. Precipitation. Precipitation has been recorded since 1948 at a US Weather Bureau gage in East Haven. Average annual precipitation at East Haven is about 45.1 inches, distributed quite uniformly throughout the year, and averaging about 3.75 inches per month. Average and extreme monthly precipitation values are shown in table C-2.

d. Snowfall. Snowfall in the Bridgeport area averages about 35 inches per year, occurring primarily between late November and March. Mean monthly snowfall recorded at the Bridgeport National Weather Service station since 1899 is presented in table C-3.

e. Rainfall Frequencies. Short duration, intense rainfall often accompanies fast moving frontal systems, thunderstorms, and coastal storms. Peak storm rainfall frequency-duration data for East Haven, summarized in U.S. Weather Bureau Technical Paper 40, are presented in table C-4.

TABLE C-1

MONTHLY TEMPERATURE IN DEGREES FAHRENHEIT
BRIDGEPORT, CONNECTICUT
(Elevation 10 Feet NGVD, 98 Years of Record)

Month	Mean	Maximum	Minimum
January	29.4	68	-14
February	29.8	70	-20
March	37.7	85	1
April	48.0	97	9
May	57.1	95	26
June	67.5	99	34
July	73.1	103	44
August	71.4	101	38
September	65.2	98	32
October	54.7	90	20
November	43.7	80	8
December	32.4	67	-12
Annual	**50.8**	**103**	**-20**

TABLE C-2

MONTHLY PRECIPITATION IN INCHES
EAST HAVEN, CONNECTICUT
(Elevation 30 feet NGVD, 45 Years of Record)

Month	Mean	Maximum	Minimum
January	3.87	14.22	0.45
February	2.99	5.76	0.62
March	4.25	11.46	0.90
April	4.40	10.03	1.39
May	3.53	8.29	0.67
June	3.26	16.93	0.11
July	3.53	8.27	0.32
August	3.97	11.20	0.54
September	2.90	9.40	0.15
October	3.95	11.41	0.88
November	4.08	8.64	1.64
December	3.43	6.26	0.64
Annual	**45.09**	**55.74**	**34.69**

TABLE C-3

MEAN MONTHLY SNOWFALL IN INCHES
BRIDGEPORT, CONNECTICUT
(Elevation 10 Feet NGVD, 95 Years of Record)

Month	Mean
January	8.9
February	10.1
March	6.7
April	1.1
May	0.0
June	0.0
July	0.0
August	0.0
September	0.0
October	Trace
November	1.4
December	6.5
Annual	**34.7**

TABLE C-4

RAINFALL FREQUENCY DURATION
USWB TECHNICAL PAPER 40
EAST HAVEN, CONNECTICUT
(Inches)

Annual Frequency		Duration in Hours					
(Return Period)		1	2	3	6	12	24
50%	(2-year)	1.3	1.7	1.9	2.4	2.7	3.4
20%	(5-year)	1.8	2.2	2.5	3.1	3.5	4.3
10%	(10-Year)	2.1	2.6	2.8	3.5	4.1	5.0
2%	(50-Year)	2.7	3.4	3.7	4.5	5.5	6.2
1%	(100-Year)	3.0	3.7	4.2	5.1	6.1	7.1

4. TIDAL HYDROLOGY

a. Astronomical Tides. In the study area, tides are semidiurnal, with two high and two low waters occurring during each lunar day (approximately 24 hours, 50 minutes). The resulting tide range is constantly varying in response to relative positions of the earth, moon, and sun, with the moon having the primary tide producing effect. Maximum tide ranges occur when orbital cycles of these bodies are in phase. A complete sequence of tide ranges, known as a tidal epoch, is repeated over an approximate interval of 19 years. Because of continual variation in water level due to tides, several reference planes, called tidal datums, have been defined to serve as reference points for measuring elevations of both land and water. The most recent epoch for which the National Ocean Survey (NOS) has published tidal datum information is 1960 thru 1978. Tidal datums are referenced periodically to the National Geodetic Vertical Datum of 1929 to account for apparent sea level rise, a phenomenon observed through tide gaging and tidal benchmark measurements along most of the U.S. coast.

Tidal reference datums and range of tide information for East Haven, shown in table C-5 and figure C-1, were developed from 1995 NOS predicted tide tables, and information developed in NED 1988 tidal flood profiles. The mean range and mean spring range of tides are 6.2 and 7.1 feet, respectively.

TABLE C-5

EAST HAVEN TIDAL DATUM PLANES
(Based Upon 1960-78 NOS Tidal Epoch)

	Tide Level (ft NGVD)
Maximum Predicted Astronomical High Water	7.5
Mean Spring High Water	4.2
Mean High Water (MHW)	3.8
Minimum Predicted Astronomical High Water	2.7
Mean Tide Level (MTL)	0.7
National Geodetic Vertical Datum (NGVD)	0.0
Maximum Predicted Astronomical Low Water	-2.4
Mean Low Water (MLW)	-2.4
Mean Spring Low Water (MLWS)	-2.9
Minimum Predicted Astronomical Low Water	-7.1

TIDAL DATUM PLANES

EAST HAVEN, CONNECTICUT

NOT TO SCALE

BASED ON NOS TIDAL BENCH MARK DATA FOR
1960 - 1978 TIDAL EPOCH

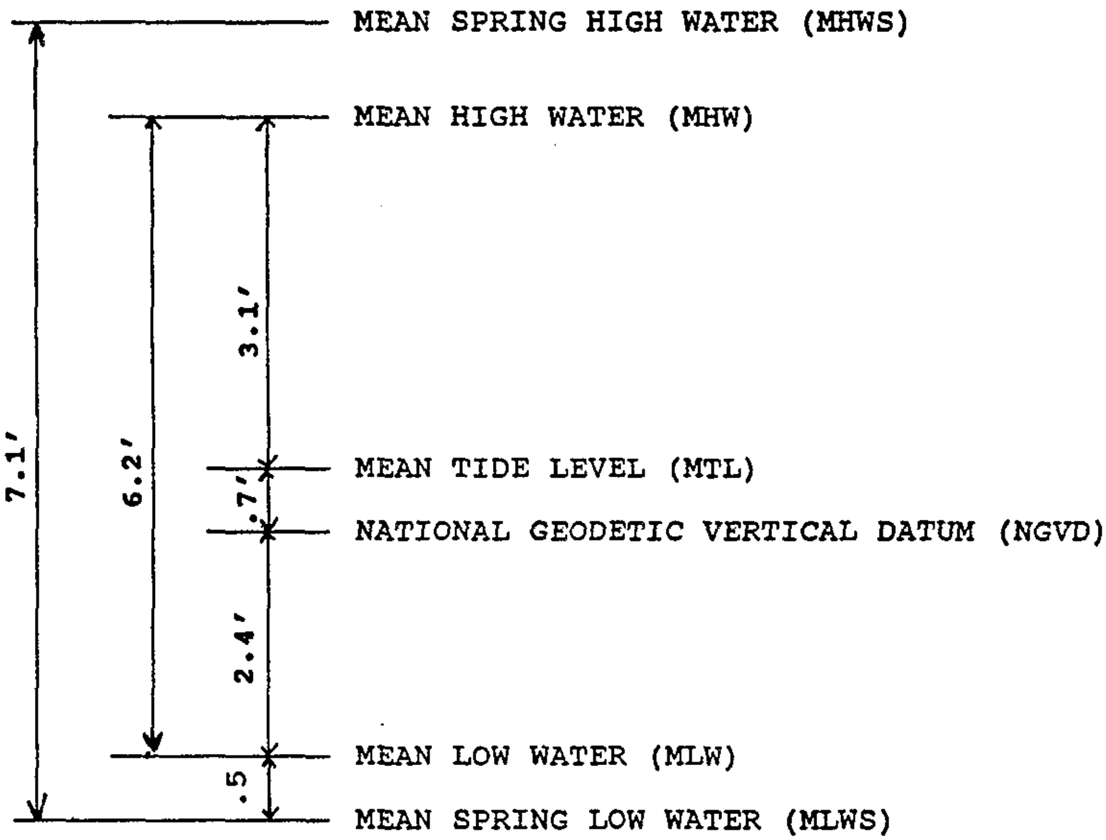


FIGURE C-1

b. Storm Types. Two distinct types of storms, distinguished primarily by their place of origin as being extratropical and tropical cyclones, influence coastal processes in New England. These storms can produce above normal water levels and waves, and must be recognized in studying New England coastal problems.

(1) Extratropical Cyclones. These are the most frequently occurring variety of cyclones in New England. Low pressure centers frequently form or intensify along the boundary between a cold, dry continental air mass and a warm, moist marine air mass just off the coast of Georgia or the Carolinas, and then move northeastward more or less parallel to the coast. These storms derive energy from the temperature contrast between cold and warm air masses. The organized circulation pattern associated with this type of storm may extend for 1,000 to 1,500 miles from the storm center. The wind field in an extra tropical cyclone is generally asymmetric with the highest winds in the northeastern quadrant. When the storm center passes parallel and to the southeast of the New England coastline, and highest onshore wind speeds are from the northeast, these storms are called "northeasters" or "nor'easters" by New Englanders. As the storm approaches and passes, local wind directions may vary from southeast to slightly northwest. Coastlines exposed to these winds can experience high waves and extreme storm surges. Such storms are the principal tidal flood producing events throughout the study area. Other storms, taking a more inland track, can have high winds from the southeast and are referred to as "southeasters." November through April is the prime season for severe extratropical storms in New England.

(2) Tropical Cyclones. These storms form in a warm moist air mass over the Caribbean and waters adjacent to the West Coast of Africa. The air mass is nearly uniform in all directions from the storm center. Energy for the storm is provided by the latent heat of condensation. When the maximum windspeed in a tropical cyclone exceeds 75 mph, it is labeled a hurricane. Wind velocity at any position can be estimated, based upon the distance from the storm center and forward speed of the storm. The organized wind field may not extend more than 300 to 500 miles from the storm center. Recent hurricanes affecting New England generally have crossed Long Island Sound and proceeded landward in a generally northerly direction. However, hurricane tracks can be erratic. Hurricanes have been a principal cause of tidal flooding in the Long Island Sound area. The hurricane and tropical storm season in New England generally extends from August through October.

(3) Tidal Flood History. Tidal flooding within the study area is caused primarily by hurricanes and extratropical storms. Hurricanes usually generate higher surge values, but their area of influence is more concentrated so the impacted geographical area is smaller. Also, hurricanes impacting New England are generally fast moving, with forward speeds of 30 knots (35 mph) and faster. In contrast, extratropical storms are generally slow moving, more extensive in area, and impact the coastline for an extended period, giving high surge values through several tidal cycles (see storm tracks of selected major hurricanes and storms on figures C-2 and C-3, respectively). Following are descriptions of the more damaging extratropical storms and hurricanes in this century.

(a) Hurricane of 21 September 1938. Damage caused by tidal flooding from this hurricane was the greatest ever experienced in Long Island Sound. The center of the hurricane entered Connecticut perpendicular to the coast about 15 miles east of New Haven or about 30 miles west of New London on 21 September, and then proceeded northwesterly at a forward speed of 50 to 60 mph. The peak of the hurricane tide arrived about 1 to 2 hours before the predicted normal high tide throughout most of the Sound, causing extreme tide levels. The tidal surge at Bridgeport was about 7 feet higher than the predicted tide. Tidal surge reached its crest at 7:30 p.m. on the 21st. The total precipitation ranged from 2 to 5 inches along the Connecticut shore. Wave heights ranged from 10 feet at New London to 15 feet at New Haven and Bridgeport.

(b) Hurricane of 14-15 September 1944. The eye of this storm passed inland just west of Point Judith, Rhode Island, and continued in a northeasterly direction. The hurricane tide arrived in the Sound about mean tide at the eastern end, and about two hours after predicted high tide at the western side, resulting in moderately high ocean levels. Precipitation during this hurricane followed a high antecedent condition and totalled five inches.

(c) Extratropical Storm of 25-26 November 1950. The 25 November 1950 storm started as a disturbance from Virginia, intensified rapidly, and moved north-northeastward reaching New England on the 25th, resulting in the most violent storm of its kind on record. Tidal flooding was experienced along the entire Connecticut coast and was particularly severe west of New Haven. The two crests of severe tidal flooding, approximately equal in height, occurred on two successive tide cycles. At New Haven, the recorded maximum one-minute sustained wind velocity and gust were 55 and 77 miles per hour, while

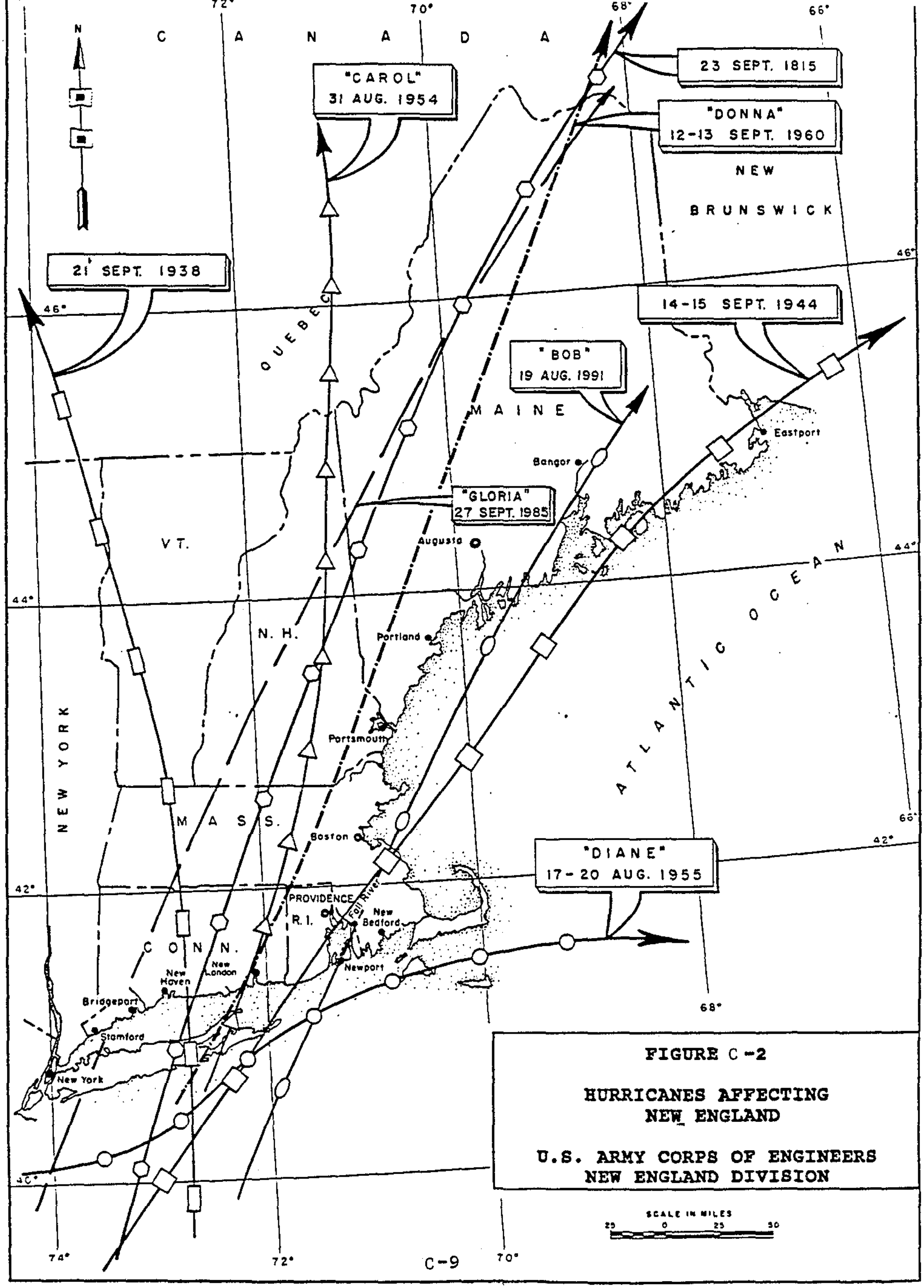


FIGURE C-2

HURRICANES AFFECTING NEW ENGLAND

U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION

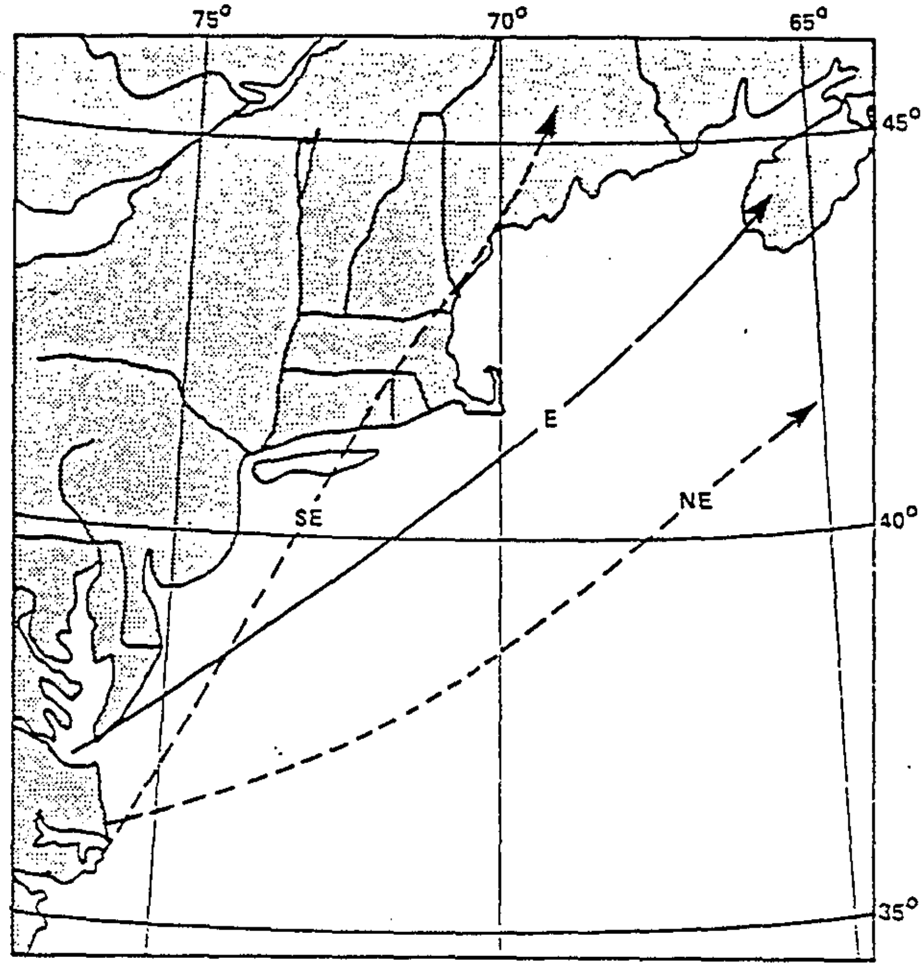


FIGURE C.-3

MEAN TRACKS OF SURGE-PRODUCING EXTRATROPICAL STORMS

U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION

corresponding velocities at Hartford were 70 and 100 mph. The wind shifted slowly from east to southeast, then south, thus becoming directly onshore at all locations along the Connecticut coast during the storm. The strong gale wind velocities were of longer duration than the 1938 and 1944 hurricanes. Although the lower limit of hurricane wind velocity was exceeded, it was not classified as a hurricane since it was not of tropical origin.

(d) Extratropical Storm of 6-7 November 1953. The 6 to 7 November 1953 storm commenced during the early morning of the 6th when a low pressure area off the Georgia coast moved rapidly up the Atlantic seaboard, developed into a major storm, bringing in rain, snow, and high winds to northeastern United States. It reached southern New England the night of the 6th, moving northerly the following day over the rest of the section. The most severe tidal flooding occurred on the 7th, coincident with the first predicted high tide for the day. Maximum wind gusts at Block Island, Rhode Island, were 98 mph. Although the entire New England coast was affected to some degree by this storm, damage was heaviest along the Connecticut shore of Long Island Sound.

(e) Hurricane of 31 August 1954 ("Carol"). The second most severe hurricane to strike southern New England in over 300 years occurred 16 years after the record 1938 event. The center of this storm crossed the shoreline of Connecticut near New London, with a forward speed of about 45 mph, and then followed a general northerly path across New England. As the hurricane surge occurred at or near predicted normal high tide within the Sound, tide levels rose to near record heights. Tidal surges ranged from 5 to 8 feet higher than predicted tides. The total 2-day rainfall during hurricane "Carol" (31 August 1954) in the Bridgeport-Stratford area was only 1.72 inches.

(f) Extratropical Storm of 6 February 1978. A storm characterized as "the blizzard of 1978" developed off the Carolina coast on 6 February, and moved slowly northward along the Atlantic seaboard where it stalled for nearly 12 hours on the 7th. Accompanying winds gusted to over 50 miles per hour in New York and Connecticut, with gusts of 79 and 92 mph at Boston and Chatham, Massachusetts, respectively. Maximum wind gusts at the Stamford hurricane barrier were 45 miles per hour from the north-northeast. About two feet of snow covered the State as this "granddaddy" northeast coastal storm passed through the area. Tides 3 to 4 feet above normal caused extensive flooding and wave damage.

(g) Hurricane of 27 September 1985 ("Gloria"). Hurricane "Gloria" made landfall in Westport, Connecticut at 12:15 p.m, after crossing Long Island Sound. The eye of the hurricane then continued on its north-northeastward track, passing near Hartford before exiting the State at Suffield about 1:13 p.m. Wind gusts of hurricane force ripped through the southern and central sections, as well as the eastern portion of Connecticut, with peak gusts for the entire State recorded at 92 mph at Bridgeport. The lowest sea level pressure was 28.5 inches at Bridgeport. Other peak wind gusts included 82 mph at Hartford, 75 at New Haven, and 66 at Windsor Locks.

Thousands of trees were uprooted across the State, taking down miles and miles of electrical wires; in many cases, power was not restored for several weeks. Along the coast, up to 20,000 people were evacuated from their homes from Greenwich to Stonington, and hundreds of small craft were torn from their moorings, damaged or destroyed. Five docks were ripped up in Milford Harbor, and about 100 pleasure craft were torn from their moorings. However, coastal flooding was at a minimum despite tides 2 to 4 feet above normal, since "Gloria" reached the coast near low tide.

(h) Hurricane of 19 August 1991 ("Bob"). Hurricane "Bob" caused high winds and heavy rain across the State of Connecticut as its center moved rapidly through southeast Rhode Island and southeastern Massachusetts; wind gusts on Block Island reached 125 mph. Some peak wind gusts recorded by the National Weather Service in Connecticut included 63 mph at New Haven and 52 mph at Bridgeport. The southeast portion of the State experienced near hurricane force winds with gusts about 70 mph from the east. High winds caused thousands of trees to fall on power lines, resulting in power outages that affected up to 300,000 customers Statewide. Along the coast, a number of small boats sank, broke from their moorings, or were damaged when beached ashore. The storm surge at the Stamford barrier was 5.7 feet. Heavy rainfall of 4 to 6 inches fell in less than 12 hours causing street flooding.

(i) Extratropical Storm of 30-31 October 1991. A low pressure system formed in the Atlantic, southeast of Nova Scotia on 28 October, and intensified as it moved west toward the New England coast. This system absorbed the remains of hurricane "Grace," which was drawn north from a position east of Bermuda. An unusually strong high pressure system was centered north of New England. The high combined with the Atlantic low produced a powerful "Halloween nor'easter" with winds gusting to 60 mph. The combination produced nearly a 13,000 mile fetch of gale and storm force winds (32 to 72 mph) from Newfoundland, Canada, to Florida.

The storm was extremely intense, causing flooding, erosion, and damage along most of the eastern New England shoreline. The duration of higher than predicted water levels was 90 hours (7.5 tidal cycles). The Halloween storm occurred simultaneously, with a period of normal astronomic tides, sparing many coastal areas greater damage.

Coastal areas in the State of Connecticut experienced flooding of coastal roads. U.S. Route 1 at Groton was closed for four hours, and Sikorsky Memorial Airport at Bridgeport was closed for several hours during early morning hours of the 31st, at the time of high tide. Scattered power outages occurred throughout the State of Connecticut due to falling tree limbs.

(j) Extratropical Storm of 11-12 December 1992. The great Nor'easter of December 1992 affected the entire population of the northeastern quarter of the United States in some way. In the State of Connecticut, this Atlantic nor'easter resulted in $60 million in insured damages, primarily from tidal inundation and water damage.

This major winter storm struck the Connecticut coast with easterly gales. Gusts up to 60 mph in coastal areas, combined with high astronomical tides, produced heavy surf and coastal flooding along the western Connecticut coast, where the 11 December morning tide was five feet above normal. The long fetch of easterly winds piled water up in western Long Island Sound to levels not observed since hurricane "Carol" in 1954. Tide gage data for Bridgeport measured the highest recorded ocean still water level of 9.3 feet NGVD at Bridgeport harbor. This is one tenth of a foot higher than the tide gage reading during the 1938 hurricane, although mean sea level, due to the rising sea level phenomenon, is now approximately four-tenths of a foot higher. Storm tides continued to run 3 to 4 feet above normal on Saturday, and 2 to 3 feet above normal on Sunday along the western Connecticut coast. The hurricane barrier at Stamford was operated for six consecutive high tides.

Hardest hit locations in Connecticut were the coastal counties of Fairfield and New Haven. Early reports indicated that 26 residences were destroyed, 1,328 units suffered major damage, and almost 1,600 units experienced minor damage. Fairfield and New Haven Counties were hardest hit, including coastal sections of Fairfield, Stratford, Milford, and East Haven. Scores of people were stranded and had to be rescued by boats, helicopters, and large trucks. There were some narrow escapes; a number of those rescued were treated for hypothermia. Hundreds of people were evacuated to local shelters. The coastal flooding caused the following types of damage to public facilities: broken seawalls, undermined and washed out roads, exposed, cracked, and undermined sewage lines, damaged electrical systems of sewage pumping stations, and eroded beaches. The storm deposited tremendous amounts of sand and silt upon local roads.

(k) Historical Data for East Haven. High watermarks are available in East Haven for the December 1992 storm. Behind West Silver Sands Beach, the stillwater elevation reached 9.2 feet NGVD; the ocean stillwater level was estimated as 9.4 feet NGVD.

c. Storm Tides. The total effect of astronomical tide, combined with storm surge, is reflected in actual tide gage measurements. Historic measurements, when available, are the best way to determine the extreme tide level and frequency of occurrence. Hurricanes, tropical storms, northeasters, or any recurring intense storms usually represent the most severe storm damage potential. These storms are usually associated with extremely large waves or high surge levels. The tide gages used to systematically record tide heights are located in shelters to eliminate the effects of wave heights, and, therefore, represent the still water elevation at that location.

Storm surge is an increase in water level above the normal astronomic tide due to a combination of wind setup, low barometric pressure, and offshore bathymetric contours, and is reflected in actual tide gage measurements. Wind setup is the vertical rise in the stillwater level of a body due to friction of winds blowing over the surface of the water, causing a rise in water level by forcing water towards the shore. An intense low atmospheric pressure condition may also cause water surface levels to rise. During most storms, low barometric pressure causes a rise in water level proportional to the magnitude of the low pressure, which could be a foot or more. The offshore bathymetric profile impacts storm surge when a constant volume of water, moving toward the shore, is forced upward by shallow or constricted bottom contours, raising the water surface elevation.

Highest water levels during a storm occur when the storm surge combines with high tide, although the storm surge itself may persist through several tidal cycles. The normal tidal elevation is available from the predicted tide or actual measurements. Obviously, a storm surge occurring at a low astronomical tide would not produce as high a water level as one occurring at a higher tide. Another variable in the makeup of the total flood tide is the variation with time of the storm surge itself.

Damage caused by tidal flooding during the 21 September 1938 hurricane was the greatest ever experienced in Long Island Sound. Peak of the hurricane tide arrived about 1 to 2 hours before the predicted normal high tide throughout most of the Sound, causing extreme tidal levels. The tidal surge at Bridgeport was about 7 feet higher than the predicted tide.

Several other hurricanes or tropical storms have caused major tidal flooding along Long Island Sound, namely, hurricane "Carol" in 1954 and the December 1992 Nor'easter, which caused the highest observed water level at Bridgeport, but only the 43rd highest observed water level at New London.

(1) Bridgeport. Since 1968, the National Ocean Survey (NOS) has systematically recorded tide heights at Bridgeport Harbor, Connecticut. The record prior to that time was developed from (a) high watermarks (NED), staff gage (Bridgeport Harbor Master), recording tide gage (NED and NOS) and data gathered at Bridgeport, (b) NOS recording tide gage data from New London related to Bridgeport, and (c) historical account at Stamford related to Bridgeport.

Table C-6 presents maximum observed stillwater tide heights through 1993 (measurements taken in protected areas, where waves are dampened out). Also, stillwater tide heights are shown with an adjustment applied to account for effects of rising sea levels through 1993. Several hurricanes or tropical storms have caused major tidal flood levels at Bridgeport. Although the extratropical storm is the prevalent type of storm affecting the study area, the tropical storm poses the greatest threat of extremely severe tidal flooding.

(2) Study Area. In the New England Coastline Tidal Flood Survey (NED, September 1988), storm tides in the study area are close to, but slightly higher than those observed at the NOS tide gages in Bridgeport and New London Harbor, where the National Ocean Survey has systematically recorded tide heights since 1965 and 1939, respectively.

TABLE C-6

MAXIMUM STILLWATER ELEVATIONS

AT BRIDGEPORT HARBOR

Day	Month	Year	Observed Elevation	Adjusted* Elevation
21	9	1938	9.2	9.6
31	8	1954	9.2	9.5
11	12	1992	9.3	9.3
14	9	1944	8.8	9.1
25	11	1950	8.8	9.1
7	11	1953	8.6	8.9
31	10	1991	8.6	8.6
12	9	1960	8.2	8.4
25	10	1980	8.2	8.3
14	10	1955	7.9	8.2
12	11	1968	7.8	8.0
13	4	1961	7.7	7.9
29	3	1984	7.9	7.9
6	3	1962	7.7	7.9
27	9	1985	7.8	7.9
25	12	1978	7.6	7.7
22	10	1988	7.6	7.6
4	4	1973	7.4	7.6
16	2	1958	7.3	7.5
9	1	1978	7.4	7.5
31	10	1947	7.2	7.5
19	2	1972	7.3	7.5
26	12	1969	7.3	7.5
12	5	1959	7.2	7.4
14	10	1977	7.3	7.4
2	12	1974	7.3	7.4
9	12	1973	7.1	7.3
22	12	1972	7.1	7.2
29	12	1966	7.0	7.2
27	11	1940	6.8	7.2
29	11	1945	6.8	7.1
22	3	1980	7.0	7.1
2	1	1987	7.0	7.1
10	1	1977	6.9	7.1
3	12	1991	7.0	7.0
20	11	1972	6.9	7.0
7	10	1972	6.8	6.9
4	12	1990	6.8	6.9
16	3	1976	6.7	6.8
16	2	1964	6.6	6.8
9	8	1976	6.7	6.8
28	1	1979	6.7	6.8
19	10	1989	6.7	6.8
22	12	1983	6.7	6.8
26	10	1943	6.4	6.8
12	2	1985	6.7	6.7
16	9	1971	6.6	6.7
23	2	1971	6.6	6.7
6	12	1984	6.6	6.7
7	5	1967	6.5	6.7
2	4	1939	6.3	6.7
5	11	1970	6.5	6.7
17	3	1956	6.4	6.7
2	12	1942	6.3	6.7
5	12	1957	6.4	6.7
17	1	1965	6.4	6.6

** Observed stillwater elevations adjusted for* sea level rise to 1993

AT NEW LONDON HARBOR

Day	Month	Year	Observed Elevation	Adjusted* Elevation
21	Sep	1938	9.7	10.1
31	Aug	1954	8.9	9.2
25	Nov	1950	6.7	7.0
14	Sep	1944	6.2	6.5
12	Sep	1960	6.0	6.2
7	Nov	1953	5.9	6.2
12	Nov	1968	5.5	5.7
7	Oct	1991	5.6	5.6
23	Jan	1987	5.4	5.4
19	Feb	1972	5.2	5.4
29	Dec	1966	5.1	5.3
	Feb	1960	5.0	5.2
12	Nov	1947	4.9	5.2
27	Oct	1985	5.1	5.2
19	Aug	1991	5.1	5.1
3	Mar	1942	4.7	5.1
4	Mar	1973	4.9	5.0
	Nov	1944	4.6	4.9
22	Oct	1988	4.9	4.9
22	Mar	1977	4.8	4.9
25	Dec	1978	4.8	4.9
16	Feb	1958	4.6	4.9
30	Nov	1963	4.6	4.8
25	Jan	1979	4.7	4.8
16	Mar	1956	4.5	4.8
	Dec	1942	4.4	4.8
9	Mar	1961	4.5	4.7
	Sep	1961	4.5	4.7
	Mar	1958	4.5	4.7
7	Mar	1962	4.5	4.7
	Dec	1962	4.5	4.7
3	Nov	1951	4.3	4.6
	Jan	1966	4.4	4.6
6	Mar	1943	4.2	4.6
	Nov	1972	4.4	4.6
22	Nov	1945	4.2	4.5
16	Oct	1955	4.2	4.5
17	Dec	1970	4.3	4.5
29	Dec	1959	4.2	4.4
4	Dec	1990	4.4	4.4
10	Nov	1990	4.4	4.4
29	Mar	1984	4.3	4.4
11	Dec	1992	4.3	4.3
23	Apr	1993	4.3	4.3
7	May	1967	4.1	4.3
26	Dec	1969	4.1	4.3
16	Dec	1974	4.1	4.2
16	Mar	1976	4.1	4.2
27	Nov	1940	3.8	4.2
31	Dec	1948	3.8	4.1
21	Nov	1983	4.0	4.1
3	Dec	1986	4.0	4.1
7	Apr	1971	3.8	4.0
25	Oct	1980	3.8	3.9
22	Nov	1952	3.6	3.9
29	May	1948	3.5	3.8

** Observed stillwater elevations adjusted for* sea level rise to 1993

Note: Estimated stages at study reaches are based on tide-stage frequency relationship developed for Bridgeport Harbor and New London, using a composite of (a) a Pearson type *III distribution function for analysis of historic and systematically observed annual max*imum stillwater tide levels, and (b) a graphical solution utilizing Weibull plotting positions for partial duration series data, using the record adjusted for sea level rise to 1993.

d. Tide-Stage Frequency

(1) Bridgeport. Release of a new NOS report on sea level rise in 1988, corrections by NOS to previous tide gage data, and occurrence of two major coastal storms in 1991 and 1992 prompted NED to reanalyze its previous systematic tide stage-frequency developed 1988. A tide stage-frequency relationship for Bridgeport Harbor, including the October 1991 and December 1992 storms, was developed using a composite of (a) a Pearson type III distribution function for analysis of historic and systematically observed annual maximum stillwater tide levels, and (b) a graphical solution utilizing Weibull plotting positions for partial duration series data (EM 1110-2-1412, 15 April 1986).

(2) Study Area. Tidal flood frequency tables, representative of open tidal stillwater conditions, were prepared for East Haven using existing information, including the 1988 NED Tidal Flood Profiles and correlation of the Bridgeport frequency analyses to East Haven. The New England Coastline Tidal Flood Survey was developed in September 1988 by statistical determination of tidal stage frequency relationships at various gaged points along Long Island Sound, and historical high watermark data at intervening points (see location map and profile, plates C-1 and C-2, respectively). The estimated tidal flooding frequency at East Haven, is summarized in table C-7 and figure C-4.

5. TIDAL HYDRAULICS

a. General. Coastal events may be linked to a combination of events such as local wind-driven waves, ocean swells, extremely high tides, and high riverflows in adjacent coastal streams. Flooding is a common effect of coastal storms due to superposition of tides, surges, winds, and waves, coupled with erosion of beaches and dunes. Storm erosion is primarily caused by waves, and higher water levels associated with storms. Storm damage can be caused by any combination of wind, waves, water levels, and intense rainfall, resulting in physical damage to structures and other facilities. The severity of storm erosion or damage is related to the length of time the higher energy waves occur, in conjunction with elevated water levels. The most important storm conditions affecting erosion or damage at a given location are wave height, period and direction, and height and duration of storm surge.

The neighborhood behind West Silver Sands Beach receives some flooding from overtopping and some from Caroline Creek.

TABLE C -7

	A	B	C	D	E	F	G	H	I	J	K
1	Ocean Stillwater Elevations, FT., NGVD - Adjusted to 1993 - Based on File CTCL872.WK1 (1988 Tidal Flood Profiles)										
2											
3	Data	Bridgeport	EAST Haven	Branford Harbor	Sachem Head	Falkner Island	Madison Beach	Duck Isle	Saybrook Jetty	New London	Data
4											
5											
6	Statute Mile	42.0	54.00	58.00	64.25	68.00	72.00	78.25	84.20	97.0	Statute Mile
7	New 100 year	10.19	10.70	10.57	10.34	10.23	10.07	9.93	9.86	9.82	New 100 year
8	New 50 year	9.72	10.10	10.00	9.82	9.73	9.60	9.37	9.17	8.74	New 50 year
9	New 10 year	8.58	8.70	8.51	8.19	8.02	7.80	7.44	7.10	6.39	New 10 year
10	New 1 year, Annual	5.74	5.57	5.46	5.23	5.27	5.01	4.75	4.20	3.76	New 1 year, Annual
11	New 1 year, Composite	6.60	6.26	6.09	5.77	5.76	5.44	5.09	4.45	3.83	New 1 year, Composite
12	NOS 60-78 Mean Tide Level	0.70	0.69	0.68	0.68	0.67	0.67	0.54	0.41	0.42	NOS 60-78 Mean Tide Level
13	NOS 60-78 Mean Spring HW	4.55	4.24	4.08	3.78	3.77	3.47	3.14	2.51	1.92	NOS 60-78 Mean Spring HW
14	NOS 60-78 Mean high water	4.07	3.79	3.63	3.38	3.37	3.12	2.79	2.16	1.71	NOS 60-78 Mean high water
15	NOS 60-78 Mean low water	-2.67	-2.41	-2.27	-2.02	-2.03	-1.78	-1.71	-1.34	-0.87	NOS 60-78 Mean low water
16	1938 Hurricane	9.20	9.50	9.47	9.40	9.38	9.32	9.34	9.42	9.70	1938 Hurricane
17	1954 Hurricane	9.20	9.50	9.47	9.40	9.38	9.32	9.20	9.10	8.90	1954 Hurricane
18	TFP'88 100 year	10.10	10.60	10.50	10.31	10.22	10.09	9.99	9.96	10.00	TFP'88 100 year
19	TFP'88 50 year	9.60	10.00	9.92	9.78	9.72	9.61	9.42	9.25	8.90	TFP'88 50 year
20	TFP'88 10 year	8.50	8.60	8.42	8.12	7.96	7.75	7.40	7.08	6.40	TFP'88 10 year
21	TFP'88 Mean Spring HW	4.55	4.24	4.08	3.78	3.77	3.47	3.14	2.51	1.92	TFP'88 Mean Spring HW
22	TFP'88 Mean HW	4.08	3.79	3.63	3.38	3.37	3.12	2.79	2.16	1.71	TFP'88 Mean HW
23	TFP'88 Mean low water	-2.68	-2.41	-2.27	-2.02	-2.03	-1.78	-1.71	-1.34	-0.87	TFP'88 Mean low water
24	FEMA 100-year Level	10.00	10.70	10.66	10.60	10.55	10.50	10.40	10.27	10.00	FEMA 100-year Level
25											
26	'38 H- TFP'88 100-yr	-0.90	-1.10	-1.03	-0.91	-0.84	-0.77	-0.65	-0.54	-0.30	'38 H- TFP'88 100-yr
27	'38 H- 1993 100 yr	-0.99	-1.20	-1.10	-0.94	-0.85	-0.75	-0.59	-0.44	-0.12	'38 H- 1993 100 yr
28											
29	New 100 - TFP'88 100	0.09	0.10	0.07	0.03	0.01	-0.02	-0.06	-0.10	-0.18	New 100 - TFP'88 100
30	New 100 - FEMA	0.19	0.00	-0.09	-0.26	-0.32	-0.43	-0.47	-0.41	-0.18	New 100 - FEMA
31											
32	TFP'88 100 - FEMA	0.10	-0.10	-0.16	-0.29	-0.33	-0.41	-0.41	-0.31	0.00	TFP'88 100 - FEMA

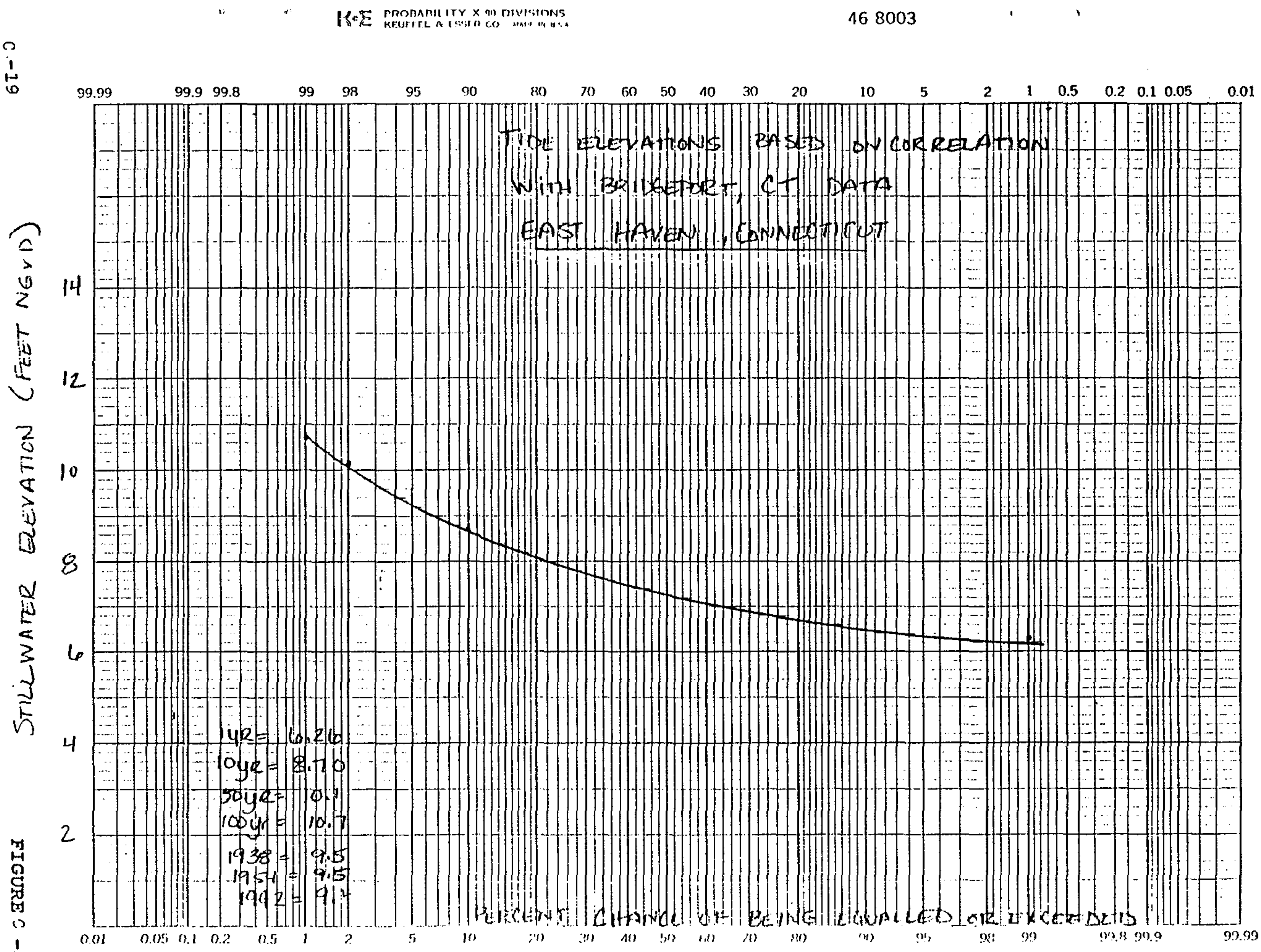

K&E PROBABILITY X 90 DIVISIONS KEUFFEL & ESSER CO.
46 8003
TIDE ELEVATIONS BASED ON CORRELATION
WITH BRIDGEPORT, CT DATA
EAST HAVEN, CONNECTICUT
STILLWATER ELEVATION (FEET NGVD)
PERCENT CHANCE OF BEING EQUALLED OR EXCEEDED
1YR = 6.26
10yr = 8.70
50yr = 10.1
100yr = 10.7
1938 = 9.5
1954 = 9.5
1962 = 9.4
99.99 99.9 99.8 99 98 95 90 80 70 60 50 40 30 20 10 5 2 1 0.5 0.2 0.1 0.05 0.01
0.01 0.05 0.1 0.2 0.5 1 2 5 10 20 30 40 50 60 70 80 90 95 98 99 99.8 99.9 99.99
2 4 6 8 10 12 14

FIGURE C-4

For this reason, alternatives for flood protection included house raising and a combination of continuing the revetment along the back of the houses, in conjunction with raising roads and a berm along West Silver Sands Beach. To evaluate these two alternatives, the extent of flooding from each source and their combined effect had to be determined. This was accomplished by performing overtopping analyses on West Silver Sands Beach, transmitting wave crest elevations through the backshore using FEMA's WHAFIS program, and simulating Caroline Creek with UNET, a one-dimensional hydrodynamic computer model.

b. Overtopping Analysis. As part of the reconnaissance investigation for flood protection of West Silver Sands back shore area, peak rates of overtopping were computed. These numbers were used to determine the frequency where overtopping begins and flooding starts to occur from the beach and also to estimate proposed berm elevations. This analysis required information on the physical geometry of the beach, winds, wave heights, and runup values.

(1) Beach Profile. An average beach profile and crest elevation were taken from 2-foot contour maps provided by the town. West Silver Sands Beach has an approximate slope of 1 on 10, and a crest elevation of 8.4 feet NGVD.

(2) Design Wave Heights, Periods, and Runup. Wave heights, periods, and runup were computed for overtopping analyses using the Automated Coastal Engineering System (ACES). Wave heights were based on fully developed open water waves generated on Long Island Sound, by winds sustained from various directions and durations, using previously analyzed National Weather Service data at Sikorsky Memorial Airport (Tidal Flood Management, West-Central Connecticut, NED 1988). Design wave heights and periods for 2-, 5-, 10-, 50-, and 100-year return periods were determined, using fetch lengths for each wind direction. This information was used to estimate proposed beach elevations to prevent overtopping for 10-, 50-, and 100-year return periods. Table C-8 presents data computed for the worst case scenario - winds from the southwest.

Wave setup was also estimated to account for the effective increase in water surface elevation at the shoreline due to onshore transport of water by wave action and breaking waves. When wave action and storm surge coincide, wave setup is added to the stillwater flooding. This does not necessarily mean that the winds must be onshore for this to occur; it is possible for some refracted waves and deep water generated waves to be striking in an onshore direction when the winds are from a different direction.

(3) Proposed Berm. Coastal Engineering Branch determined that a sandfill profile with a 50 foot wide berm, a seaward slope of 1 on 15, and a landward slope of 1 on 5 would be the most suitable for this area. The berm top elevation was determined from wave height and runup calculations for 10-,50-, and 100-year events. These elevations are shown in table C-8.

TABLE C-8

WAVE HEIGHT, PERIOD, AND RUNUP

Frequency (yrs)	SWL* (ft NGVD)	Wind-Speed (mph)	Wave Height (ft)	T (sec)	Proposed Berm Elevation (ft NGVD)
2	7.4	20	3.9	5.0	
5	8.2	23	4.7	5.4	
10	8.8	24	4.9	5.4	12.0
50	10.3	29	6.3	5.9	13.5
100	11.0	31	6.8	6.1	15.5

- Average height from Rayleigh distribution
- Existing beach slope 1:10
- Fetch = 32 miles, winds from SW
* SWL includes wave setup

(4) Peak Overtopping Rate

(a) General. The Automated Coastal Engineering System (ACES), version 1.07e "Wave Runup and Overtopping on Impermeable Structures" was used to estimate peak rates of overtopping along the existing beach. For each particular return period, a local windspeed from the southwest direction was assumed to be occurring during the overtopping period. Overtopping coefficients were estimated, using the 1984 Shore Protection manual and best engineering judgement; see following section.

The condition when waves break at the structure toe was assumed to be critical, producing the maximum wave runup and peak rates of overtopping of the existing conditions. The structure toe is defined at the base of the beach, or the point where the structure slope intersects the near shore slope. Depth at the structure/beach toe was assumed to be the difference between tidal stillwater level for the particular return period, and elevation of the intersection of the structure and near shore slope.

(b) Overtopping Coefficients. There are two coefficients which need to be estimated for calculating overtopping using ACES- Q^*_0 and alpha. These are empirically determined, and depend on incident wave characteristics and structure geometry. Initial estimates of Q^*_0 were made using conoidal theory from figure 7-34 (Variation of Q^*_0 between waves conforming to conoidal theory and waves conforming to linear theory) in the Shore Protection Manual. This requires computing H_0/gT^2; where H_0 is the equivalent deepwater wave height, "g" is the gravitational coefficient, and "T" is the wave period. For the 2-year condition with the 3.9-foot, 5-second wave determined by Coastal Engineering Branch, H_0/gT^2 equals 0.0048. The corresponding Q^*_0 from figure 7-34 is 0.018. For the 5-year condition with the 4.7-foot, 5.4 second wave, H'_0/gT^2 equals 0.005. The corresponding Q^*_0 from figure 7-34 is 0.015. An option available in ACES was used to make initial estimates of alpha. For the 2 and 5-year existing conditions, the ACES-calculated alphas are 0.092 and 0.093, respectively for the existing beach slope of 1 vertical to 10 horizontal. The same methodology was followed for the proposed berms.

There are no known volumes of overtopping for 2- or 5-year storms in the study area for comparison with computed volumes. High watermarks are available in the area for more extreme storms such as the December 1992 event, which had an approximate frequency of 25 years.

Table C-9 presents results of the overtopping analysis along West Silver Sands Beach for existing and proposed conditions. The volumes of overtopping were used to aid in the development of stage-frequency curves presented in paragraph 6.

TABLE C-9

PEAK OVERTOPPING RATES

Freq (yrs)	SWL (ft, NGVD)	Existing Beach (cfs/lf)	Proposed 12' NGVD (cfs/lf)	Proposed 13.5' NGVD (cfs/lf)	Proposed 15.5' NGVD (cfs/lf)
2	7.4	1.4	negligible	negligible	negligible
5	8.2	5.0	negligible	negligible	negligible
10	8.8	*	negligible	negligible	negligible
50	10.3	*	0.28	negligible	negligible
100	11.1	*	4.5	0.14	negligible

- Proposed beach slope 1:15
- Existing beach slope 1:10
- Fetch = 32 miles, winds from SW

* Beach crest inundated by stillwater level; therefore, overtopping not calculated

c. Wave Crest Transmission. In order to determine an elevation for raising houses, the wave height and crest elevation, associated with the 10, 25, 50, and 100-year storm surge as the waves propagate through the area, were computed according to the Federal Emergency Management Agency's Wave Height Analyses for Flood Insurance Studies (WHAFIS). The model takes into account transformation of the wave height due to local winds, bottom interaction effects, and presence of features such as buildings, as the wave propagates along a transect.

One transect was assumed representative of the study area. Ground surface elevations along this transect were obtained from 2-foot contours provided by the town. Open space ratios of 0.62 to 0.68 were computed from the building density in the study area. In the WHAFIS computations, depth limited waves in shallow water reach a maximum breaking height equal to 0.78 times the still water depth, and the wave crest is 70 percent of the total wave height above the still water level.

A graphical depiction of the maximum wave crest analysis is depicted in figure C-5. Results of the wave crest

WAVE HEIGHT ANALYSIS CONCEPTS

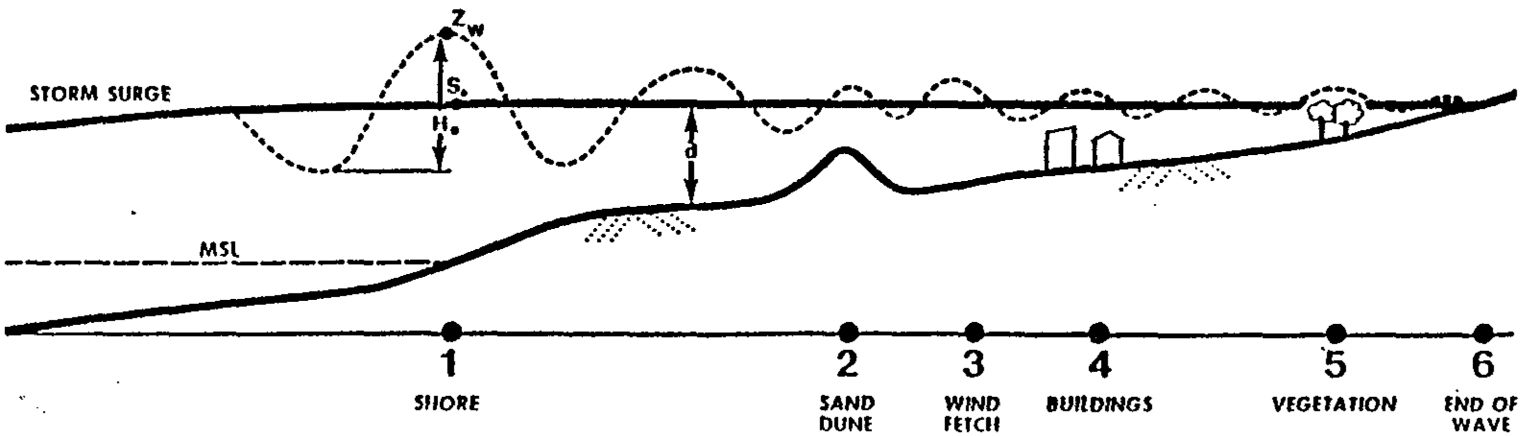


FIGURE C-5
WAVE HEIGHT ANALYSIS CONCEPTS

Source: FEMA

transmission analysis for the backshore of West Silver Sands Beach is presented in table C-10. This table shows that houses directly on the beach would need to be raised to a minimum elevation of 15.7 feet NGVD and interior houses to a minimum of 13.0 feet NGVD.

TABLE C-10

WAVE CREST TRANSMISSION

Storm Event	Maximum Wave Crest Elevation at the Beach (feet NGVD)	Maximum Crest at Interior Houses (feet NGVD)
10 year	12.2	9
25 year	13.3	11
50 year	14.8	12
100 year	15.7	13

d. Flooding Analysis of Caroline Creek. The hydraulic analysis of Caroline Creek and the marsh to the north of West Silver Sands Beach was performed using a one dimensional hydrodynamic model, UNET, the latest most advanced model readily available, which would provide reasonable results without the significant difficulties of a two-dimensional model. UNET also has the capability to more accurately model culverts since it is able to use the Federal Highway Administration procedures for determining hydraulic capacities of culverts. Three culverts were simulated in this analysis, one under Brazos Road and two under Fairview Road. This model became available for Corps use through the Hydrologic Engineering Center in September 1992, and was updated in March 1993.

UNET, using the properties of continuity and momentum, applies a linearized, implicit finite difference scheme to solve a set of linear equations. The equations are linearized, using the first order Taylor approximation. The program can simulate one-dimensional unsteady flow through a full network of open channels. For subcritical flow, stages are a function of channel geometry, and downstream backwater effects. UNET provides the user with the ability to apply flow and stage hydrographs, bridges, spillways, levee systems, and culverts. Cross sections are input in a modified HEC-2 forewater format. Input required to the model includes marsh geometry, Mannings "n", and tide information.

(1) Marsh Geometry. Two-foot contour maps provided by the town were used to develop marsh geometry for input into the computer model. The model starts at the mouth of Caroline Creek at Long Island Sound, and continues through the culverts under Brazos and Fairview Roads. Surveys to obtain channel width, depth, and inverts were performed in August 1995 by NED personnel using a hand level and rod.

(2) Manning's "n". An important input parameter into the UNET model is the roughness coefficient, Manning's "n". A Manning's "n" of 0.045 was used for the channel, which has gradual bends and appears to be slightly weedy. A Manning's "n" of 0.07 was used for the overbanks which are flat with short marsh grass.

(3) Tide Measurements. Staff gage readings and tidal measurements were not available for calibration of UNET model for this study.

(4) Results. The model was run using a mean tide range and 2-, 5-, 10-,50-, and 100-year storm events estimated from Long Island Sound. Water levels predicted by the model at the end of the existing revetment are shown in table C-11. The model indicates that, under mean tide conditions, there is little reduction in tide levels from Long Island Sound through the marsh. The existing culvert sizes under Brazos and Fairview Roads cause minor restrictions to tidal flow. The mean high tide was reduced 0.1 foot after passing through the Brazos Road culvert and an additional 0.1 foot through the Fairview Road culverts. During events with frequencies of 10 years or greater, the stillwater level in the marsh, before it reaches Brazos Road culvert, is essentially the same as in the open ocean.

TABLE C-11

WATER LEVEL IN THE MARSH
AT THE END OF THE EXISTING REVETMENT

Tide Frequency (event)	Ocean Still-water Level (ft NGVD)	Marsh Elevation (ft NGVD)
Mean tide	3.8	3.8
2 year	7.4	7.4
5 year	7.95	7.9
10 year	8.7	8.7
50 year	10.1	10.1
100 year	10.7	10.7

(e) Conclusions. Hydraulic analysis completed indicates that West Silver Sands Beach receives tidal flooding from the south (over the beach) and from the north (flow through Caroline Creek). West Silver Sands Beach begins to experience overtopping from a 2-year event and is fully inundated by a 10-year event. At the same time, tidal flow enters the backshore via Caroline Creek. Water surface elevations in the marsh at the end of the existing revetment are essentially the same as those in the open ocean; Brazos and Fairview Roads are too low to provide protection.

6. INTERIOR HYDROLOGY

a. Description of the Area. The backshore of West Silver Sands Beach, the area under study, is located in the southern part of East Haven. Development in this area consists of private residences. Ground elevations in the study area are low, ranging from 4.5 to 7.4 feet NGVD, street elevations vary between 4.7 and 5.6 feet NGVD, and the drainage area is approximately 0.25 square mile.

Due to its flat topography, the low lying neighborhood behind the beach acts as a storage area for direct rainfall, rainfall runoff, overtopping from West Silver Sands Beach, and floodwaters from the marsh.

b. Storm Rainfall. Rainfall contributes a much lower proportion of flooding in the study area. Twenty-four hour rainfall amounts and maximum hourly rainfall rates recorded at Bridgeport during the five most recent significant storm events, as reported by the National Oceanic and Atmospheric Administration, are listed in table C-12. This table shows that most of the experienced storm rainfall quantities and rates were in the order of 0.7 to 1.7 inches and 0.1 to 0.95 inches per hour, respectively. During the August 1991 event, the 24-hour rainfall amounted to a total of 4.7 inches, however, the maximum hourly rainfall was 0.95 inches. In comparison, the all-season 20 percent chance (5-year frequency) 1-hour rainfall rate is reportedly 1.8 inches (see table C-4).

c. Interior Flood Stages; Combined Flooding of Tides in Caroline Creek and Overtopping

(1) General. For frequencies greater than one year, floodwaters from the marsh inundate the backshore area at essentially the same elevation as the ocean stillwater level. During 2- to 5-year storm events, overtopping from West Silver Sands Beach increases interior water levels

TABLE C-12

RECENT FLOODS AT BRIDGEPORT, CT
COMPARATIVE HYDROLOGIC DATA

Flood Event	31 Aug 1954	6-7 Feb 1978	27 Sept 1985	19 Aug 1991	11-12 Dec 1992
Observed Ocean Tide (ft NGVD)	9.4	9.0	7.84	8.7	9.3
Tide Frequency, Est (%)	3.5	6	26	13	5.5
Maximum 1 hr Rainfall (inches)	0.45	0.12	0.3	0.95	0.15
Storm Rainfall (inches)	1.62/24 hr	1.1/48 hr	0.62/24 hr	4.7/24 hr	1.7/48 hr

above ocean stillwater. Two methods were used to evaluate the combined flooding effects of Caroline Creek and overtopping of West Silver Sands Beach. The first used a starting interior water level estimated by UNET, added an overtopping volume from ACES, and estimated the resultant interior flood stage using an area-capacity curve. The second was to rerun UNET, with an overtopping inflow hydrograph.

(2) Area-Capacity Method. An area-capacity curve was developed for existing conditions to evaluate the combined flooding effects of Caroline Creek as analyzed by the UNET model, and overtopping as evaluated with ACES. Curves were estimated from 2-foot contour mapping and other limited information, such as spot elevations. Total storage capacity of the study area was estimated from a starting water surface elevation in the interior of 7.4 feet NGVD (from UNET) as overtopping began. Using the area-capacity curve, the computed overtopping volume was added to the starting water surface elevation, and a flood level was determined.

(3) UNET Method. The increase from beach overtopping was evaluated by inputting the overtopping volume as a flow hydrograph into the UNET model.

(4) Results. Interior water levels generated by both methods for 2- and 5-year events, shown in table C-13, are very close.

TABLE C-13

INTERIOR WATER SURFACE ELEVATIONS

Tide Frequency (event)	Area-Capacity Elevation (feet NGVD)	UNET Elevation (feet NGVD)
2 year	7.8	7.8
5 year	8.35	8.3

Using the area-capacity curve, UNET output, and computed overtopping volumes, the stage-frequency curve, shown in plate C-4, was developed. For frequencies greater than 10 years, the beach is inundated and the interior water level is the same as the ocean stillwater level. For frequencies less than 10 years, floodwaters from the marsh inundate the backshore area at essentially the same elevation as the ocean stillwater level. Overtopping, occurring during 2- to 5-year events, increases the water level to that above the ocean stillwater.

d. Modified Conditions. Possible alternatives for this study include house raising, or continuation of the existing revetment along Caroline Creek to meet Brazos Road and raising the road, in conjunction with a berm on West Silver Sands Beach at either 12, 13.5 or 15.5 feet NGVD to provide protection against 10-, 50-, and 100-year events, respectively.

A new area-capacity curve was developed for these alternatives because the ponding area, bordered by the revetment, raised road, and elevated beach, is much smaller than existing.

Stage-frequency curves for the alternatives using a 50- and 10-year beach are shown on plates C-5 and C-6, respectively. A berm along West Silver Sands Beach at elevation 15.5 feet NGVD would prevent most waves from a 100-year event from overtopping. The berm, in combination with a continued revetment along Caroline Creek and the raising of Brazos Road, would essentially eliminate flooding of the backshore of West Silver Sands Beach. A berm at 13.5 feet NGVD would eliminate overtopping from events up to and including 50-year frequencies. Overtopping would still occur from a 100-year event. A berm at 12.0 feet NGVD would only eliminate overtopping from events up to a 10-year frequency.

7. *CONCLUSIONS*

The purpose of this study was to determine the amount of flooding caused by overtopping from West Silver Sands Beach and from high tides in Caroline Creek and to use this data to estimate elevations for house raising, revetments, raising roads, and the proposed berm.

Wave crest transmission analyses, using WHAFIS, indicate that to protect against a 100-year storm, houses directly on the beach would have to be raised to at least 15.7 feet NGVD, while interior houses would need to be raised to at least 13 feet NGVD.

Wave height, runup, and overtopping analyses using ACES generated berm elevations of 12.0, 13.5, and 15.5 feet NGVD to protect against 10-, 50-, and 100-year events, respectively. UNET was used to estimate revetment and road elevations. The model indicates that ocean stillwater levels are not decreased within the marsh. The revetment and road would therefore need to be at 11.0 feet NGVD to protect against a 100-year storm.

Once elevations for alternatives were determined, stage-frequency curves were developed for evaluating the benefits of each alternative.

CORPS OF ENGINEERS

U. S. ARMY

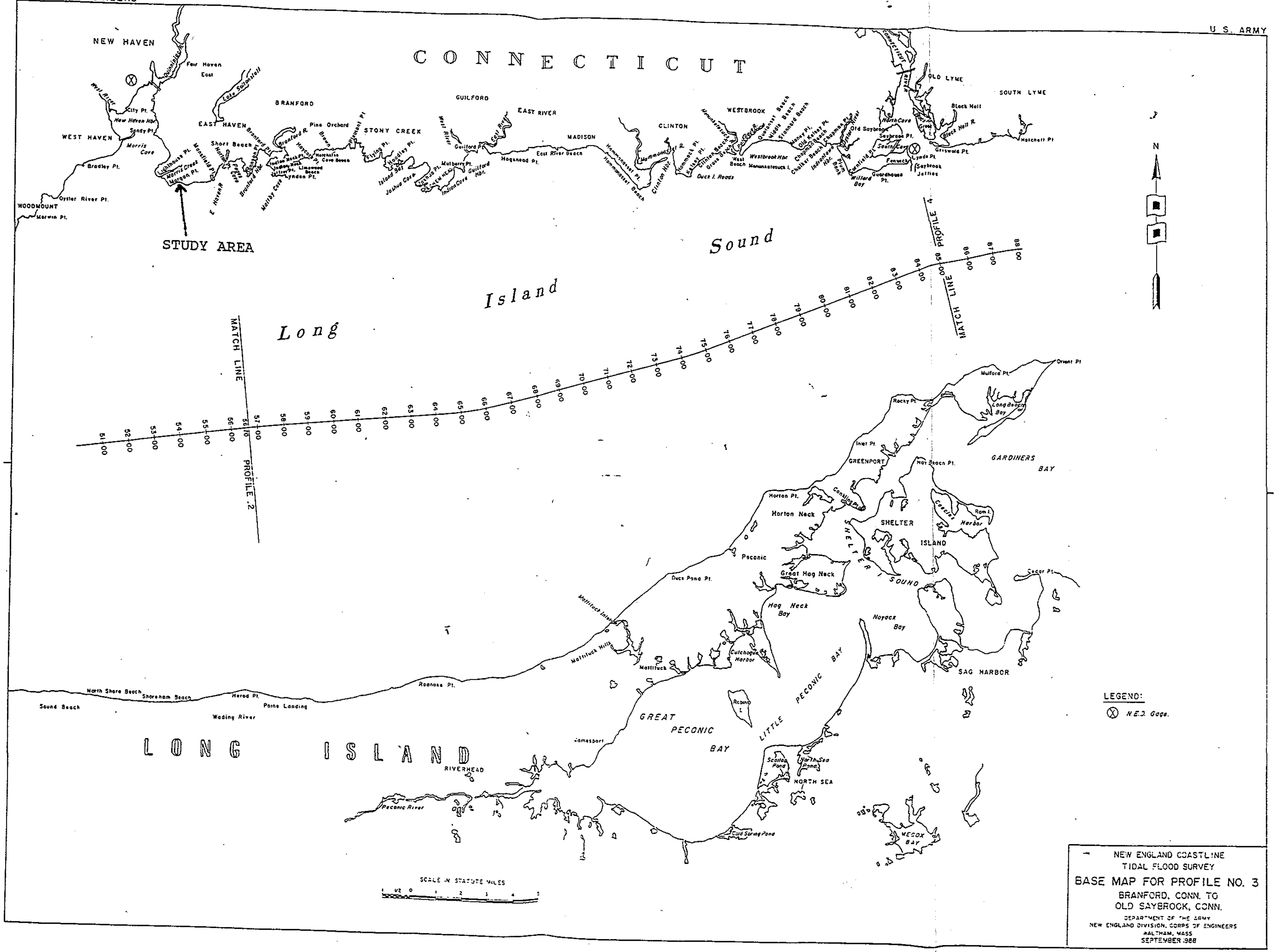
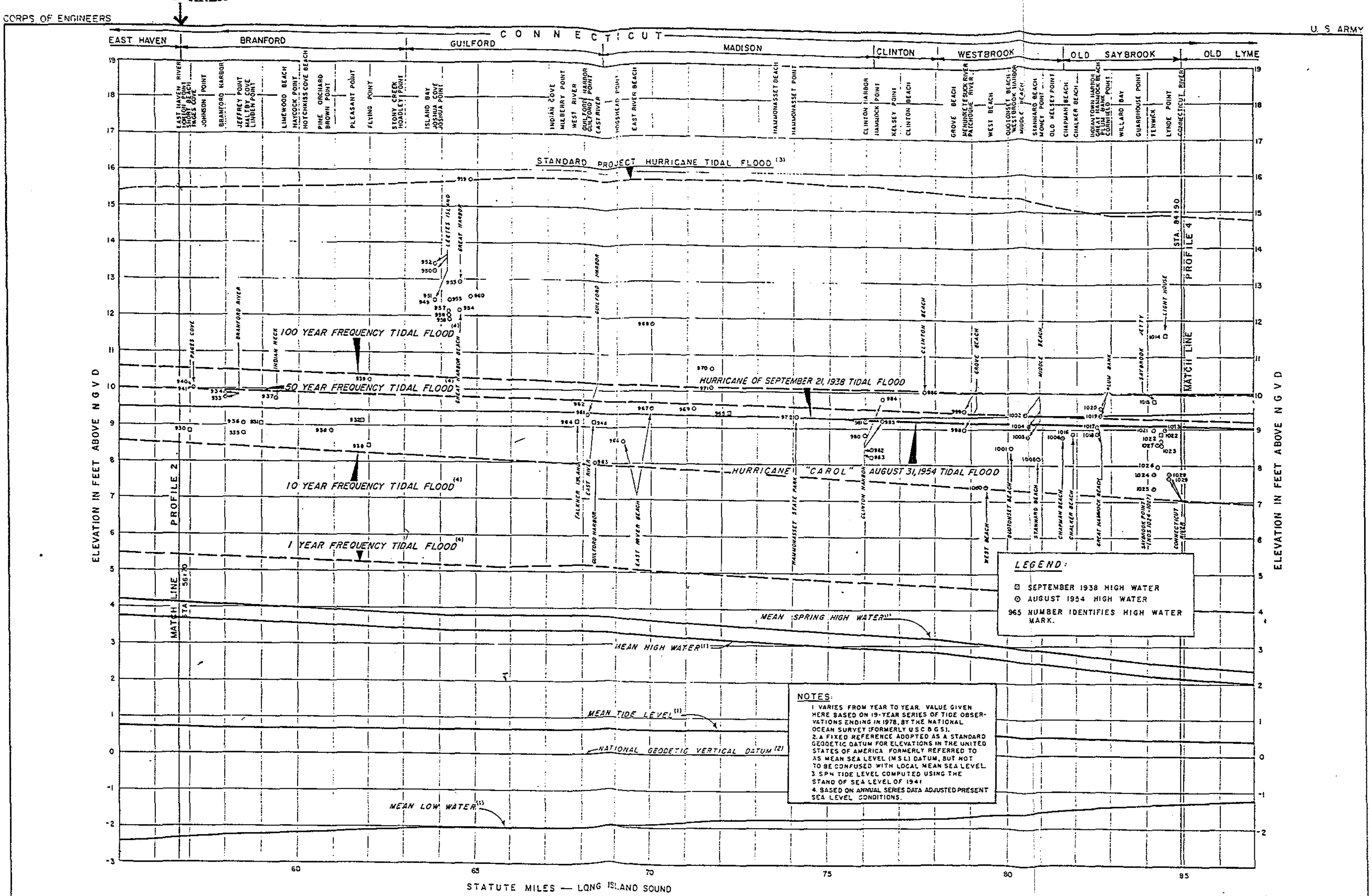


PLATE C-1

STUDY AREA



NEW ENGLAND COASTLINE
TIDAL FLOOD SURVEY

TIDAL FLOOD PROFILE NO. 3

BRANFORD, CONN., TO
OLD SAYBROOK, CONN.

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS
SEPTEMBER 1988

PLATE C-2

WEST SILVER SANDS BEACH
STUDY AREA

K&E PROBABILITY X 90 DIVISIONS KEUFFEL & ESSER CO. MADE IN U.S.A. 46 8003

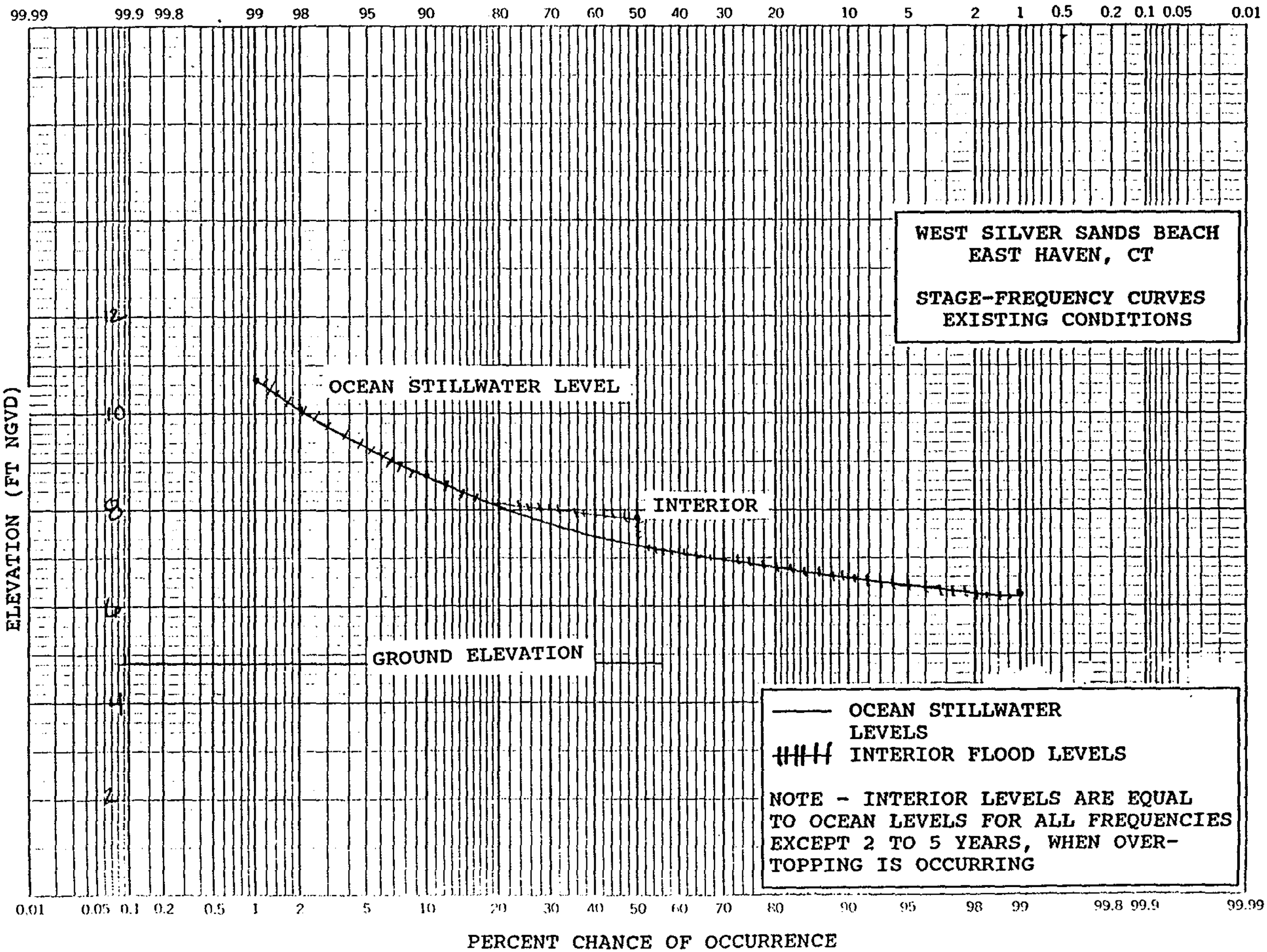
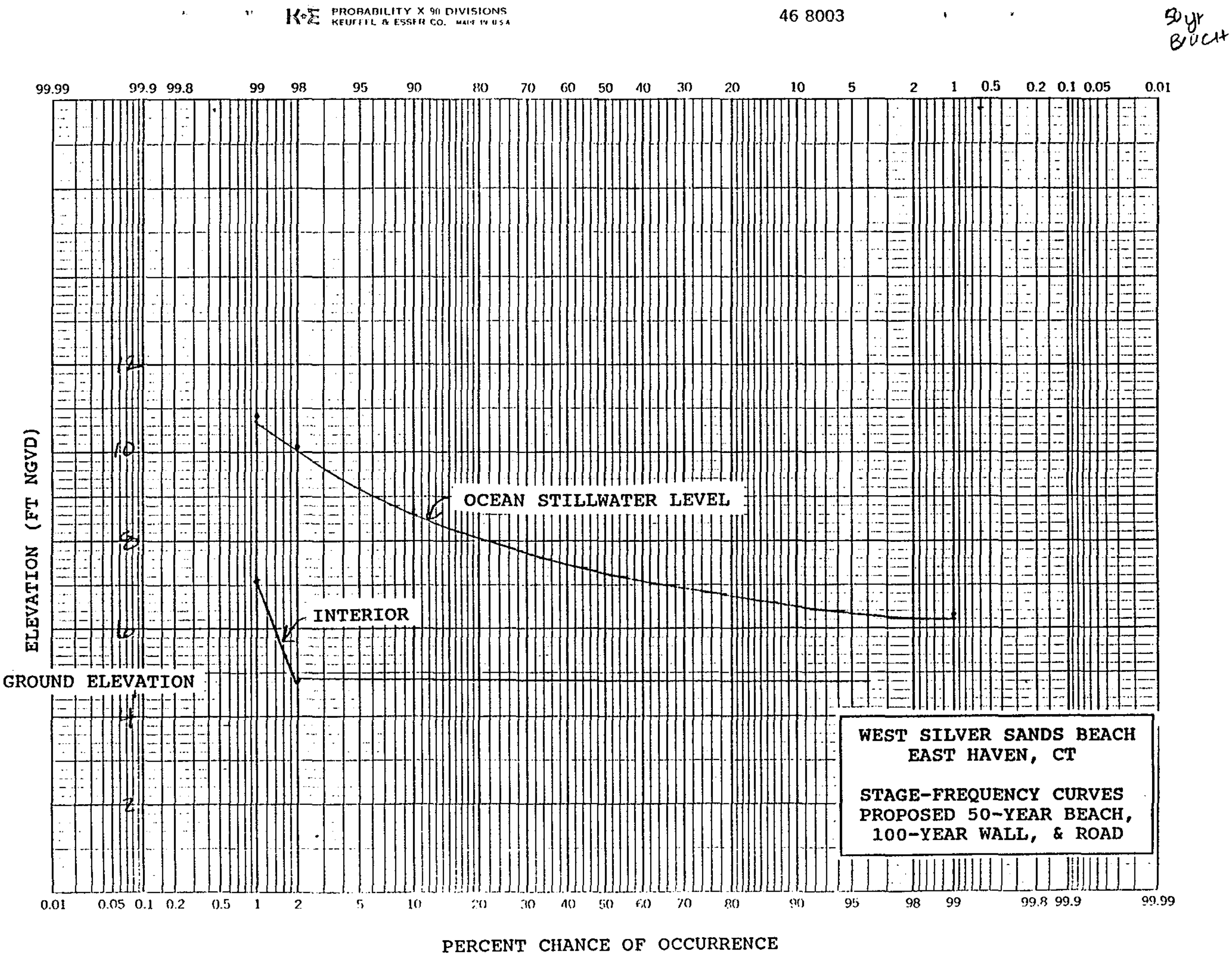


PLATE C -4



K&E PROBABILITY X 90 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8003
50 yr
BUCH
99.99
99.9
99.8
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.5
0.2
0.1
0.05
0.01
ELEVATION (FT NGVD)
12
10
8
6
4
2
OCEAN STILLWATER LEVEL
INTERIOR
GROUND ELEVATION
WEST SILVER SANDS BEACH
EAST HAVEN, CT
STAGE-FREQUENCY CURVES
PROPOSED 50-YEAR BEACH,
100-YEAR WALL, & ROAD
0.01
0.05
0.1
0.2
0.5
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99
99.8
99.9
99.99
PERCENT CHANCE OF OCCURRENCE

PLATE C-5

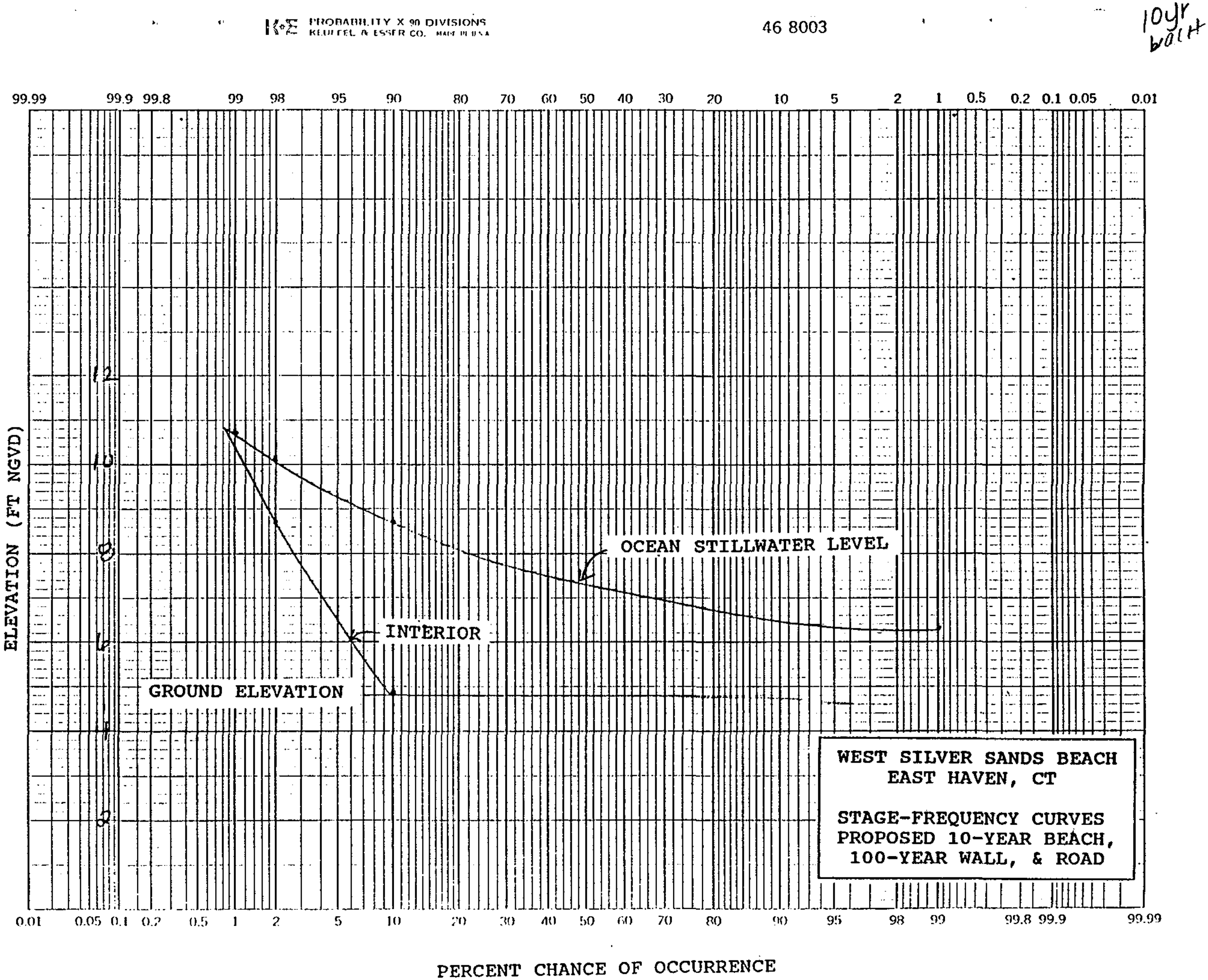

K+E PROBABILITY X 90 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8003
10yr wall
ELEVATION (FT NGVD)
OCEAN STILLWATER LEVEL
INTERIOR
GROUND ELEVATION
PERCENT CHANCE OF OCCURRENCE
WEST SILVER SANDS BEACH
EAST HAVEN, CT
STAGE-FREQUENCY CURVES
PROPOSED 10-YEAR BEACH,
100-YEAR WALL, & ROAD

PLATE C-6

APPENDIX D
STRUCTURE RAISING AND COST ANALYSES

STRUCTURE RAISING ANALYSIS

STRUCTURE RAISING ANALYSIS

The 100-year flood zone elevations were determined from FEMA Flood Insurance Rate Maps. A 100-year flood elevation of 13 feet NGVD (V zone) was used for Ellis Road, Caroline Road, Fairview Road and a portion of Brazos Road. A 100-year flood elevation of 11 feet NGVD (A zone) was used for four homes at the inland end of Brazos Road (attachment A). These elevations were defined by Planning Directorate to be the elevations below which homes may be economically eligible for house raising. First floor elevations were approximated using a clinometer, leveling rod and a two-foot contour topographic map.

The 51 structures within the study area on the contour map include the following: 43 residences, 3 garages, 2 storage sheds, 1 pump house, and 2 locations where structures are indicated but no longer exist (attachment B). None of the residences had attached garages. The majority were closely-spaced seasonal cottages with footprints of less than 1,000 square feet. The 43 residences were classified for cost estimating purposes according to the following structure types:

Type A - One- or two-story timber structures with a footprint of less than 1,000 square feet and without masonry chimneys.

Type B - One- or two-story timber structures with a footprint greater than 1,000 square feet, with an odd or complex shape, or with a masonry fireplace.

All 43 residences examined have first floor elevations below the FEMA 100-year flood elevations. Of these 43, 28 were classified as type A structures and the remaining 15 were classified type B.

The flood zone within which the house is located may have an impact on house raising costs. Foundations within the V zone typically require replacement with structures which allow waves to flow beneath the elevated home and which are designed to withstand the effects of impact and scour from waves. The area beneath a V zone home is also required under the National Flood Insurance Program to remain free of obstructions to flow. Less stringent requirements for A zone homes allow the extension of existing foundations. The majority of the homes (39 of 43) are located within the V zone.

WEST SILVER SANDS BEACH FEASIBILITY STUDY
HOUSE RAISING
East Haven, CT

ATTACHMENT A

STREET	1ST FLOOR ELEVATION (FT NGVD)	FEMA 100 YEAR ELEVATION	STRUCTURE TYPE
3 Ellis Road	10.5	13	A
5 Ellis Road	10.5	13	A
6 Ellis Road	7.7	13	A
7 Ellis Road	10.9	13	A
8 Ellis Road	10.3	13	A
10 Ellis Road	10.2	13	B
11 Ellis Road	9.8	13	A
13 Ellis Road	9.8	13	A
27 Ellis Road	9.8	13	A
5 Caroline Road	11.5	13	A
8 Caroline Road	10.6	13	B
10 Caroline Road	11.5	13	B
12 Caroline Road	11.3	13	A
25 Caroline Road	10.0	13	A
27 Caroline Road	10.1	13	B
34 Caroline Road	10.0	13	A
35 Caroline Road	9.8	13	A
39 Caroline Road	9.8	13	A
40 Caroline Road	11.7	13	B
42 Caroline Road	12.1	13	A
46 Caroline Road	9.1	13	B
47 Caroline Road	12.2	13	B
50 Caroline Road	11.2	13	A
56 Caroline Road	11.5	13	B
58 Caroline Road	11.3	13	B
62 Caroline Road	12.1	13	A
64 Caroline Road	11.7	13	B
68 Caroline Road	12.9	13	A
80 Caroline Road	11.0	13	B
82 Caroline Road	9.9	13	A
84S Caroline Road	10.6	13	B
84N Caroline Road	11.8	13	A
86 Caroline Road	11.3	13	A
101 Caroline Road	11.8	13	B
5 Fairview Road	10.6	13	A
10 Brazos Road	10.5	13	B
14 Brazos Road	9.2	13	B
18 Brazos Road	8.7	13	A
22 Brazos Road	9.2	11	A
26 Brazos Road	10.1	11	A
32 Brazos Road	9.7	11	A
429 Brazos Road	9.8	11	A

TOTAL HOMES IN STUDY	**43**
# TYPE A	**28**
# TYPE B	**15**

ATTACHMENT B

COST ANALYSIS

WEST SILVER SANDS BEACH
EAST HAVEN, CONNECTICUT

Designed By: CENED-ED
Estimated By: CENED-ED-C

Prepared By: Karen Schofield

Preparation Date: 02/08/96
Effective Date of Pricing: 06/09/95

Sales Tax: 0.00%

LABOR ID: CT9501 EQUIP ID: NAT93A Currency in DOLLARS CREW ID: NAT95A UPB ID: NAT95A

** 2ND VIEW SUMMARY **

			** CONTRACT **		
	QUANTITY	UOM	UNIT	TOTAL	PROJECT
10 Dike and Concrete I-Wall					
10.01 Mob, Demob & Preparatory Work	1.00	LS	18904.80	18,905	23,802
10.91 Construction of Dike	1125.00	LF	549.09	617,725	777,752
10.92 Concrete I-Wall - Seaside OPT#1	300.00	LF	427.64	128,292	161,527
10.93 Concrete I-Wall - Seaside OPT#2	162.00	LF	234.12	37,927	47,753
10.94 Concrete I-Wall - Inner Section	665.00	LF	374.69	249,168	313,717
10 Dike and Concrete I-Wall	2252.00	LF	467.15	1,052,016	1,324,551
21 Full Beach Coverage - 10 Year					
21.01 Mob, Demob & Preparatory Work	1.00	LS	12603.20	12,603	15,868
21.70 Beach Fill	189000.00	CY	20.67	3,907,011	4,919,160
21 Full Beach Coverage - 10 Year	3000.00	LF	1306.54	3,919,614	4,935,029
22 Full Beach Coverage - 50 Year					
22.01 Mob, Demob & Preparatory Work	1.00	LS	12603.20	12,603	15,868
22.70 Beach Fill	257000.00	CY	20.67	5,312,708	6,689,017
22 Full Beach Coverage - 50 Year	3000.00	LF	1775.10	5,325,311	6,704,885
23 Full Beach Coverage - 100 Year					
23.01 Mob, Demob & Preparatory Work	1.00	LS	12603.20	12,603	15,868
23.70 Beach Fill	355000.00	CY	20.67	7,338,566	9,239,693
23 Full Beach Coverage - 100 Year	3000.00	LF	2450.39	7,351,169	9,255,561
24 Limited Beach Coverage - 10 Year					
24.01 Mob, Demob & Preparatory Work	1.00	LS	12603.20	12,603	15,868
24.70 Beach Fill	90000.00	CY	20.67	1,860,481	2,342,457
24 Limited Beach Coverage - 10 Year	1650.00	LF	1135.20	1,873,085	2,358,326
25 Limited Beach Coverage - 50 Year					
25.01 Mob, Demob & Preparatory Work	1.00	LS	12603.20	12,603	15,868
25.70 Beach Fill	124000.00	CY	20.67	2,563,330	3,227,386
25 Limited Beach Coverage - 50 Year	1650.00	LF	1561.17	2,575,933	3,243,254
26 Limited Beach Coverage- 100 Year					
26.01 Mob, Demob & Preparatory Work	1.00	LS	12603.20	12,603	15,868
26.70 Beach Fill	174000.00	CY	20.67	3,596,931	4,528,751
26 Limited Beach Coverage- 100 Year	1650.00	LF	2187.60	3,609,534	4,544,619
30 House Raising					
30.31 Bldgs, Residence: A-Zone, Type A	4.00	EA	30831.57	123,326	155,275
30.32 Bldgs, Residence: V-Zone, Type A	24.00	EA	32504.09	780,098	982,190
30.33 Bldgs, Residence: V-Zone, Type B	15.00	EA	35647.50	534,713	673,235
30 House Raising	43.00	EA	33445.05	1,438,137	1,810,701

The cost estimate is for a reconnaissance study for West Silver Sands Beach in East Haven, Connecticut. The study involves the analysis of three alternatives and their associated costs. Alternative 1 is the construction of a Dike and Concrete I-Wall. Alternative 2 involves several options of beach fill protection. Alternative 3 is a house raising project.

Alternative 1 includes three parts, with one part having two possibilities being considered. Part one is a 1125 linear feet dike under Brazos Road. Part two is the more inland 665 linear feet concrete wall on sheet-piling. Part three is the seaside concrete wall which is 462 linear feet and has two possibilities, a less than 12' to bedrock and a greater than 12' to bedrock. For part three, 2/3 of the total length was assumed to be greater than 12 feet to bedrock and 1/3 was assumed to be less than 12 feet to bedrock to be conservative.

Alternative 2 considered three levels of protection for two options, a full beach coverage and a limited beach coverage. The three levels of protection considered were for 10-, 50-, and 100-year storms.

Alternative 3 involved raising houses higher off the ground. There are 43 houses proposed to be raised. The necessary raising appears to range from 1 foot to 5 feet above the current heights in order to be above the 100-year flood level. Four of the houses are A-zone, Type A which means the 100-year elevation is 11 feet NGVD and the house is a one- or two-story timber structure with a footprint of less than 1,000 square feet and without a masonry chimney. Thirty-nine houses are within the V-zone which means the 100-year flood elevation is 13 feet NGVD. Of them, 24 are classified as Type A (same as above) and 15 are classified as Type B which applies to a one- or two-story timber structure with a footprint greater than 1,000 sqaure feet, with an odd or complex shape, or with a masonry fireplace. For this study models based on the Point Beach Project in Milford, CT were used.

Labor rates for this estimate were taken from the Connecticut General Decision Number CT950001 for Middlesex County, Modification #3, dated 6/09/95. An escalation factor was also applied to each alternative with the assumption that the middle of construction would be the summer of 1998. The escalation indices were taken from the Civil Works Construction Cost Index System (CWCCIS) EM 1110-2-1304, dated 02 January 1996.

This preliminary construction cost estimate includes standard percentages for contractor overheads, profit and bond. A 15 percent contingency has also been applied to each alternative to account for the preliminary level of design and unknowns.

The attached summary page shows the contract price and unit prices which include the direct construction cost and the contractor's overheads, profits and bonds. The summary also shows the project cost which includes the escalation and contingency costs.

1. The 15% contingency for the Levees and Floodwalls alternative was added due to the preliminary nature of the design and the unknowns regarding work in wetlands and actual geology of the area.

2. The 15% contingency for the Beach Replenishment alternative was added due to the preliminary nature of the design and the unknowns regarding construction methods to deal with filling in the water.

3. The 15% contingency for the House Raising alternative was added due to the lack of actual design and the preliminary nature of information. It also accounts for unknowns including work on patios, decks, driveways, and numbers of columns. Should also cover more detailed work on the few houses with existing foundations and a more complicated design since the models used were based on basics.

APPENDIX E
ECONOMIC ANALYSIS

West Silver Sands Beach, East Haven, Connecticut
Section 103 Feasibility Study
Economic Analysis

Introduction

The purpose of this analysis is to identify and evaluate the economic feasibility of providing flood damage protection to the Brazos and Ellis Roads area of West Silver Sands Beach in East Haven, Connecticut. The town of East Haven is located in south-central Connecticut, on the shore of Long Island Sound. West Silver Sands Beach is located in the southwestern portion of the town, on Long Island Sound and near New Haven Harbor, which separates East Haven and the city of New Haven to the west. The study area includes the portion of West Silver Sands Beach that contains Brazos Road, Ellis Road, and Caroline Road. The area experiences frequent flooding, including very frequent road flooding.

This economic analysis includes a description of the study area, estimates of the recurring and expected annual flood damages for the study area, estimation of the annual benefits that would be obtained from preventing those damages, and a determination of the economic justification of the various improvement plans examined by calculating the benefit to cost ratio of each plan.

Methodology

In a general sense, the economic benefits of a project are estimated by comparing the without project condition to the with project condition, and evaluating the difference between the two conditions. A project is considered economically justified if it has a benefit to cost ratio equal to or greater than 1.0, that is, if the benefits of the project equal or outweigh the costs of the project.

In accordance with Corps of Engineers guidelines, the benefit to cost ratios for the improvement plans examined in this Study are calculated by comparing the benefits and costs of each plan in average annual equivalent terms. Costs and benefits are converted to annual equivalents based on the fiscal year 1996 federal interest rate for water resources projects of 7 5/8 percent, and based on a 50 year period of analysis. Costs presented in this analysis are based upon June 1995 levels, benefits are stated at the 1996 price level, and the analysis is performed at the feasibility level of detail.

Description of the Study Area

West Silver Sands Beach is bordered to the east by Caroline

Creek, to the South by Long Island Sound, and to the west by Morgan Point. The study area for this Feasibility Study includes the eastern half of West Silver Sands Beach, from Caroline Creek to the western end of Caroline Road. North of the study area is a tidal marsh, fed by Caroline Creek. The study area includes the structures which are located between Long Island Sound and the tidal marsh in the backshore. The study area contains 43 houses and 2 storage shed structures. The topography of the study area is generally flat, with ground elevations around elevation 5 NGVD in the eastern half of the study area, and rising to around elevation 8 in the western half of the study area. The majority of the structures are contained in the eastern half of the study area, where the elevations are lower.

The houses in the study area are all single family houses, most of which are seasonal residences. According to residents of the study area, approximately 15 percent of the houses in the study area are occupied year-round, and the recent trend has been toward increasing year-round occupancy from seasonal occupation. The houses in the study area include one-story, one-and-one-half story, and two-story houses, most of which have no basements, and most of which are built on piles. The condition of the houses in the study area varies, including structures in excellent, good, and only fair condition. The two storage sheds in the study area are one-story, wooden structures, each containing a number of garage-type bays.

Economic Setting

Located next to the city of New Haven, the largest city in Connecticut, the town of East Haven is part of the greater metropolitan area of New Haven and shares in the economy of that area. In general, the town contains mostly residential development, with some clusters of commercial and retail development. According to the 1990 US Census, in 1990 East Haven had a total population of 26,144 and a median family income of $42,797.

Existing Conditions

The study area is subject to frequent road flooding and less frequent but still problematic flooding of the structures in the area. The area can flood from any of several sources, including heavy rains, high tides, or coastal storms. During a very high tide, the area floods from the tidal marsh behind the study area, often causing a low area on Brazos road and the low end of Ellis Road to flood. If a high tide combines with heavy rains or a coastal storm, the flooding can become so severe that the roads become impassable, causing safety hazards and access problems. In the coastal storm of December 1992, the area suffered severe flooding, with some flooding of the structures in the study area. While the area residents are concerned about the flooding of the

While the area residents are concerned about the flooding of the houses, they are even more concerned with the road flooding which happens more frequently and causes significant safety problems. Local residents estimated that the roads in the area are impassable from flooding an average of at least 20 times a year.

Without Project Condition

It is projected for the without project condition that no measures providing permanent, effective protection to the study area from flood damages will be constructed in the study area by town, state, or federal interests. As a result, for the without project condition, it is assumed that the flooding of residential structures and the very frequent road flooding that has occurred in the study area in the past will continue to occur in the future.

With Project Condition

The with project condition consists of providing coastal flood protection to the study area with a federal Section 103 project. The alternatives examined in this analysis to protect the study area include constructing a beach along the shoreline to protect the area up to a 10 year event, constructing a beach to protect the area up to a 50 year event, constructing a beach to protect the area up to a 100 year event, and elevating the first floors of the houses in the study area to an elevation above the 100 year event. Details regarding these alternatives are contained in the main report.

Inventory and Survey of Structures

As part of this study, the structures in the study area were inventoried and surveyed. The size, type, condition, and general construction of each structure was noted. The first floor elevation of each structure was estimated using available topographic mapping and based on a survey that was conducted as part of the Engineering analysis. Based on the information collected, the structures were categorized based on size, type, and whether they have basements.

Development of Stage-Damage Function for Study Area

Once all of the structures in the study area were inventoried, surveyed, and categorized, a stage-damage function for each category of structure was used to estimate the flood damages that would be likely to occur at various flood elevations at each structure. The typical stage-damage functions used had been developed for previous Corps of Engineers studies, adjusted as judged appropriate to reflect the depreciated repair and replacement costs for the structures in the study area. The damages in the typical stage-damage functions are estimated in one foot increments, from the basement, if applicable, to six feet above the first floor. The damage functions include damage estimates for structural damage, damage to contents, damage to utilities, damage to outside grounds, and estimated non-physical losses such as costs for temporary relocation during a flood. Since very few of the structures in the study area have basements, most of the typical damage functions used have damages beginning at the first floor, except for minor outside and grounds damages which can occur below the first floor.

Combining the typical stage-damage function assigned to each structure with the first floor elevations of each structure, the depth-damage function for each structure was determined. The depth-damage functions for each structure were then aggregated to determine the stage-damage function for the study area as a whole.

Recurring Losses

Recurring losses are those potential flood related losses which are expected to occur at various stages of flooding under current development conditions. As the final output of the flood damage survey process, the dollar value of losses in the project area are determined for an array of events ranging from very likely, frequent events to very rare, infrequent events. A breakdown of the estimated recurring losses by flood event in the study area are shown in Table 1, below.

Table 1
Recurring Losses
West Silver Sands Beach, East Haven, Connecticut

Probability (%)	Frequency (year)	Stage (feet)	Estimated Damages ($)
50	2	7.8	$ 15,600
20	5	8.1	$ 28,900
10	10	8.7	$ 86,600
5	20	9.3	$207,500
2	50	10.1	$465,400
1	100	10.7	$723,100

Expected Annual Damages

Expected annual damages are calculated by multiplying the predicted damages for each flood event by the annual percentage chance that each flood event will occur. The resulting expected damages at each event are then added together to yield the expected annual damages for the study area. The expected annual damage figure represents the average annual damage that could be expected to occur based on the weighted probabilities of the complete range of storm events. For this study, these calculations were made using the Corps of Engineers' SID (Structural Inventory of Damages) and EAD (Expected Annual Damages) flood damage analysis programs. The estimated expected annual damages for the study area under existing conditions were calculated to equal $40,810.

Calculation of Benefits

The effectiveness of a flood damage reduction plan is measured by the extent to which it reduces expected annual damages. In order to determine the benefits of the different improvement plans examined, the expected annual damages under existing conditions, are compared to the expected annual damages with each improvement plan. The difference between the without and with project expected annual damage figures equals the annual benefits of each improvement plan. The expected annual damages with each improvement plan were calculated using the Corps of Engineers' SID and EAD flood damage analysis programs. The results of the analysis are shown in Table 2, below. The expected annual damages (EAD) under the without project condition, the expected annual damages (EAD) under each with project condition, and the annual benefits of each plan, are all shown.

Table 2
Benefit Analysis of Improvement Plans

Alternative	EAD Exist. Cond.	EAD With Project	Annual Benefits
10-year Beach	$40,810	$9,060	$31,750
50-year Beach	$40,810	$ 30	$40,780
100-year Beach	$40,810	$ 0	$40,810
House Raising	$40,810	$5,070	$35,740

Economic Justification

In order for a federal project to be considered economically justified, it must have a benefit to cost ratio equal to 1.0 or greater. The annual benefits, annual costs, benefit to cost ratio, and net annual benefits for each alternative improvement plan examined are shown in Table 3, below. Additional information regarding the costs of the alternatives are contained in the main report.

Table 3
Economic Justification
Brazos and Ellis Roads, East Haven, Connecticut

Alternative	Annual Benefits	Annual Costs	Benefit to Cost Ratio	Net Annual Benefits
10-year Beach	$31,750	$288,100	0.11	none
50-year Beach	$40,780	$357,300	0.11	none
100-year Beach	$40,810	$459,00	0.09	none
House Raising	$35,740	$141,700	0.25	none

APPENDIX F
PERTINENT CORRESPONDENCE

STATE OF CONNECTICUT
CONNECTICUT HISTORICAL COMMISSION

December 8, 1995

Mr. Joseph L. Ignazio
Planning Directorate
Evaluation Division
Corps of Engineers
424 Trapelo Road
Waltham, MA 02254-9149

Subject: West Silver Sands Beach
East Haven, CT

Dear Mr. Ignazio:

The State Historic Preservation Office understands that the Corps of Engineers is preparing a reconnaissance report for a proposed Section 103, Shore and Beach Restoration and Protection, regarding the above-named study area. This office notes that the residential structures located in the West Silver Sands Beach Area were built primarily as 20th-century summer homes, which have been since converted to year-round occupancy. We believe that the West Silver Sands Beach Area neighborhood does not possess architectural distinction and would not warrant further cultural resource consideration.

Despite the 20th-century residential development, the State Historic Preservation Office believes that the West Silver Sands Beach Area possesses moderate to high sensitivity for prehistoric archaeological resources. The current absence of reported prehistoric sites within the study area reflects a lack of professional investigation, rather than an accurate assessment of the area's archaeological potential. Similar circumstances exist with respect to off-shore areas.

This office looks forward to further coordination with the Corps of Engineers concerning the identification, evaluation and professional management of the state's archaeological heritage vis-a-vis the National Historic Preservation Act of 1966.

For further information please contact Dr. David A. Poirier, Staff Archaeologist.

Sincerely,

Dawn Maddox

Dawn Maddox
Deputy State Historic
Preservation Officer

cc: Dr. Nicholas Bellantoni/OSA

TEL: (203) 566-3005 FAX: (203)566-5078
59 SOUTH PROSPECT ST. - HARTFORD, CONN. 06106 - 1901

June 16, 1994

Planning Directorate
Coastal Development Branch

Honorable Rosa DeLauro
Representative in Congress
One Century Tower
265 Church Street, Suite 303
New Haven, CT 06510-7004

Dear Ms. DeLauro:

I am writing in response to your letter of May 24, 1994 regarding a flooding problem in the Brazos and Ellis Roads area in the Town of East Haven. You forwarded a letter from Mayor Henry J. Luzzi requesting assistance from this agency in solving the problem.

Ms. Catherine LeBlanc of my staff is coordinating the investigation and has been in contact with Ms. Jennifer Cosenza of your staff. A site inspection has been scheduled for June 29, 1994. Based upon the site inspection, to be performed by Ms. LeBlanc and Ms. Karen Umbrell, an economist from this office, a determination will be made as to whether the conditions that would permit us to provide Federal assistance are in place.

I trust that this information meets your needs. If you have any questions, please call me at (617) 647-8222 or Ms. Catherine LeBlanc of my staff at (617) 647-8564.

Sincerely,

Dwight S. Durham
Lieutenant Colonel, Corps of Engineers
Division Engineer

Copy Furnished:

Honorable Rosa L. DeLauro
House of Representatives
Washington, DC 20515-0703

cc:
Ms. LeBlanc, 114S (BRAZOSRD)
Mr. Smith, 114S
Mr. Pronovost, 114N
XO, 100
Reading File
Plng. Dir. File, 114N
CDB File, 114S

327 Cannon Building
Washington, DC 20515-0703
(202) 225-3661

One Century Towers
265 Church Street
New Haven, CT 06510
(203) 562-3718

Killingworth/Clinton
(203) 669-1181

Stratford
(203) 378-9005

United States
House of Representatives
Rosa L. DeLauro
3d District, Connecticut

Committee on Appropriations
Subcommittees:
Labor, Health and
Human Services, Education
Agriculture

May 24, 1994

Colonel Brink P. Miller
Commander/Division Engineer
U.S. Army Corps of Engineers
New England Engineer Division
424 Trapelo Road
Waltham, Massachusetts 02154

Dear Colonel Miller:

Enclosed is the letter I recently received from Mayor Henry Luzzi on behalf of our constituent, Mr. Dan Carloni. Mr. Carloni's family owns a home in the vicinity of Brazos and Ellis Roads in East Haven.

I would appreciate it if you could review this correspondence to determine whether your staff can schedule a site visit to this area to evaluate the erosion and flooding problems. You can notify me of the outcome through my District Office located at One Century Tower, 265 Church Street, Suite 303, New Haven, Connecticut 06510-7004. If you need any additional information, please contact my staff assistant, Jennifer Cosenza, at 203-562-3718.

Thank you for your assistance with this matter.

Sincerely,

Rosa DeLauro

ROSA L. DeLAURO
Member of Congress

RLD/jac

Enclosure

•TOWN OF•
EAST HAVEN
250 MAIN STREET • EAST HAVEN, CONNECTICUT 06512 • (203) 468-3204 • FAX (203) 468-3372

MAYOR HENRY J. LUZZI
Office of the Mayor

May 23, 1994

The Honorable Rose DeLauro
265 Church St.
New Haven, CT 06510

Dear Rose,

I am writing to express my concern about the wave action and the flooding problem in the Brazos Rd. and Ellis Rd. area of East Haven, and to request an Army Corps of Engineers site visit to this area.

As always, your assistance is greatly appreciated. Thank you.

Sincerely,

Henry J. Luzzi
Mayor